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Real Reasons for Wrecks Having made up their minds as to the causes of the North Haven wreck on the New York, New Haven & Hartford Railroad, the public and the newspapers are not likely to give the attention it deserves to the finding of the coroner's jury made public on Sept. 15. Public opinion had, prior to this finding, settled down to a conviction that it was the inferior signal system of the New Haven road that was primarily responsible for the wreck. The recent wreck at Tyrone on the Pennsylvania Railroad and many others of which railroad men, at least, have full knowledge afford more than enough proof to satisfy reasonable minds that the North Haven wreck was caused by faulty organization and lack of discipline and that no system of signals will prevent wrecks if trainmen disregard these signals. The emphasis which the coroner who investigated the North Haven wreck lays upon improper flagging of the train that was struck and the passing of signals as the causes of this and other railway disasters is entirely justified. It is unfortunate both for the railways and the public that so many misleading statements and comments should be published to the effect that railroad wrecks can be avoided by what are called "the best signal systems." Signal systems are, of course, better and worse, but obviously the best signals will not prevent train collisions if engineers either do not see them or run by them. If there was any criminal negligence involved in the North Haven wreck it lies just where the coroner's verdict puts it —on the men who ran by the signals. The attempt to prove that the wreck was due to failure to install something better than the banjo-signal system falls to the ground in the face of the undisputed facts of the case.

Committee on Filter Operation The news of the creation of the new committee on water filtration and reporting of operating results announced at the meeting of the New England Waterworks Association last week will be received with satisfaction by every engineer, chemist and bacteriologist interested in filtration. The association has long been noted for the useful work of its committees, and if past results in other fields of investigation are a criterion the report of the new committee will be a valuable document. The time is ripe for just such action as has been taken. The filtration of water supplies has developed in a very few years from crude attempts at straining out dirt to a science demanding the best technical ability obtainable. No science can progress very far by uncoordinated efforts. Every year sees an increasing number of filtration works installed; scores of men are striving in one way or another to operate their plants efficiently, but thus far there has been comparatively little co-operation among filter operators either in the matter of an exchange of ideas or of uniform methods of recording results. As Prof. George C. Whipple pointed out in proposing the formation of the new committee, the practice of publishing monthly average results of filter operation followed pretty generally at present accomplishes no very useful results. What is wanted is more detailed information or data arranged according to a standard form which will allow anyone to form a reliable estimate of the dependability of the filters for producing a water of uniformly safe quality. The peaks and valleys of the operating curves—which are lost in the general averages—are really the most interesting points for the investigator, and from an analysis of them facts are disclosed which would never be uncovered in a study of general averages. Of course, the committee, intrusted as it is with a work which offers a field so rich in opportunity, can really accomplish little without the hearty co-operation of the filter manager and the water analyst. It is from the man behind the operating table that many helpful suggestions must come. From the general interest manifested at the convention the support of the members of the sanitary-engineering profession seems assured.

Design of Large Bridges The designing of standard types of engineering structures for quite ordinary conditions and loads, while frequently requiring high-grade engineering judgment, is to a large extent mechanical work. The development of a new type of structure, however, the modification of a standard type to fit unusual conditions, and the extension of dimensions far beyond ordinary limits are real engineering tasks. Designing work is too often done with an equipment limited to a more or less thorough knowledge of mathematics, science and mechanics, familiarity with standard methods and skill in the application of general types of structure. Such requirements and abilities alone qualify one as a subordinate only. The mark of the true engineer is that he be creative—that a wealth of experience backed by thorough knowledge shall have given him the vision to see beyond the usual types and methods. A paper displaying to a high degree the fruits of broad experience and the creative ability referred to is printed on page 321. It is at once a résumé of the considerations governing the determination of the design of what is to be one of the greatest structures in the world and an epitome of broad principles of modern bridge design. Mr. Modjeski's paper will be of interest to engineers far beyond the confines of structural engineering.

Courses in Valuation Work College curricula tend to follow more closely as years go by the demands of practice. In the effort to be "practical" the fundamentals, unfortunately, sometimes suffer in order that the college may keep in step with the crashing strides of industrial, commercial and governmental progress. That the pressure to add "practical" courses comes from without no less than from within is evidenced by the proposals of alumni associations and the frequency with which practitioners turn to the schools for help. A recent evidence of the demand upon the schools was the suggestion by Mr. John A. Atkinson, chairman of the Public Service Commission of Missouri, that the State university establish courses in physical valuation and appraisement of public utilities. Within limits such a suggestion will receive the hearty approval of engineers even though they will wonder where time can be found in the present crowded curricula. Courses in these subjects, when treated from the economic standpoint, are not new, but in many schools they have not been available to engineering students. While there is no doubt that this is but one of many subjects "on the waiting list," so to speak, its inclusion as an optional study in the engineering course would warrant more sympathetic consideration than is accorded the many specialties frequently proposed. It is not merely because an increasing number of graduates will find attractive employment in which knowledge of these matters will be directly useful, but because the entire complexion of public-utility management is undergoing a change on account of the influence of regulation. A fundamental knowledge, therefore, of such questions as the purposes of valuation, the theory of capital and maintenance charges, the theories as to reproduction value, and the factors entering into rate making is necessary for a clear comprehension of the position of the organizations with which so many young engineers must cast their lot.

Submarine Pipe Difficulties The laying of the Narrows siphon, which will carry water from the Catskill Aqueduct underneath New York Harbor to Staten Island, will add to the history of the great project one of its most interesting chapters. As Mr. Flinn points out in an article on page 317, the Narrows siphon, which will be built of 36-in. flexible joint pipe, will involve in an exaggerated degree all of the usual difficulties of subaqueous pipe laying and some others seldom, if ever, encountered. Physical conditions at the proposed crossing precluded the selection of a tunnel. Many engineers have expressed the belief that never before has an attempt been made to lay a submerged pipe across a channel with such dense traffic as is carried by the Narrows, for this channel is the gateway to the port of New York. It is needless to say that the pipe laying must be done without interfering with this traffic. Winds, tides, ice and fogs are factors which will demand careful consideration in the maintenance of the floating equipment. To avoid damage from ships' anchors the pipe will have to be placed in a trench with its bottom at a maximum depth of 72 ft. and then carefully covered. The design of the flexible joint follows in a general way a type which has been used rather extensively on other work, but the engineers of the Board of Water Supply have introduced several improvements, one of them involving the use of gib screws for forcing cold lead into the shrinkage spaces of the pipe joints. As is usual in contracts for work of this kind, hydrostatic tests are provided for and the leakage must not exceed 40 gal. per joint per twenty-four hours. Furthermore, the tar dip commonly used was not considered permanent and a special form of enamel was selected. All in all, the laying of this line should afford many points of interest, both to the designing and the construction engineer.

Diesel-Engine Locomotive The rapid extension of the uses of the Diesel engine would, naturally, sooner or later, bring its adaptability for locomotive driving into consideration. It is not surprising, therefore, to learn that the firm of Gebrüder Sulzer, in conjunction with Herr Diesel and Herr Klose, have not only been studying the problem but have produced a locomotive which has been in service experimentally on the Prusso-Hessian State Railway since March of this year. The details of its construction are given in a recent issue of "Engineering," London, but there are neither operating data nor information as to the difficulties, if any, met with in service. The locomotive measures 54½ ft. long over all and weighs 95 long tons. There are two pairs of drivers, with four-wheeled bogies at each end. The driving-wheel base is nearly 12 ft. The main engines consist of two pairs of cylinders inclined 90 deg. to each other, driving onto a crank shaft in the plane of the driver axles. The drive is transmitted to the wheels through connecting rods from outside disk cranks on the end of the crank shaft. The engines are of the reversible two-cycle single-acting type, with cylinders having a diameter of 15 in. and a stroke of 21.7 in. When running at 304 r.p.m. the speed of the locomotive is 62 miles per hour. A compressor engine set of 250 hp is provided for starting, having in connection with it a receiver system. The statement is made that this auxiliary has a capacity of one-fifth to one-sixth of the main engines, from which it is inferred that the main power plant has a capacity of about 1250 to 1500 hp. Besides this equipment there is an elaborate assortment of pumps for air, circulating water and oil, making the locomotive a most complicated machine, beside which the steam locomotive is simplicity itself. The journal already referred to, while welcoming the advent of experiments along this line and admitting certain advantages over the steam locomotive in desert country where water is scarce, believes that the complexity is not only a serious bar to the Diesel-engine locomotive at present but will for a long time be a heavy handicap. In its very nature, as we know the large-size internal-combustion engine now, it must be much more complicated than the steam engine. Improvements unquestionably will come if the new locomotive has decided advantages in operating economy under some given set of conditions widely prevalent, but for the present where such conditions prevail repairs to a highly complicated mechanism are so difficult to provide for properly as to overbalance the advantages.

Purchasing through the Consulting Engineer

In the present state of engineering development a good many people appear to be somewhat at sea regarding the wisdom of employing an expert adviser in connection with the purchase of machinery and supplies. This ought to be a pretty late day to set up an argument as to the benefits of a consulting engineer's services. The increasing complexity of modern industry, the interconnection of technical specialties, and the multiplying resources of the manufacturers leave no room for discussion as to the real economy of outside advice of the right kind, by the bona-fide expert, rendered to the plant owner who is by force of circumstances unable to maintain a specialized knowledge of many auxiliary matters bearing upon his business. It is a difficult enough job to keep up to the times in any branch of applied science, manufacturing or business, to-day without attempting to be one's own guide, philosopher and friend in solving stiff problems outside one's normal line of work.

On some kinds of work the routine services of a purchasing bureau are no doubt sufficient for all practical requirements, provided those are supplemented by the scrutiny of the owner's technical staff. Successful experience with given makes of apparatus not involving large individual investments per unit naturally leads to "repeat orders" on which the expenditure of any considerable sums for outside advice is often needless. It requires good judgment to know when the duplication of supplies and machinery should be called into question and expert advice sought on later products. Where the investment is large the purchaser will do well, before placing a repeat order, to ascertain whether the last type of apparatus installed represents the latest and most economical machinery available for the desired service, making a special point of determining whether the service itself may not have changed sufficiently to warrant fresh engineering consideration of the best methods of meeting the demands. This may at times be done within one's own organization with entire success, while in other cases it must be done by the aid of outsiders. In a nutshell, the retention of a consulting engineer in every case should depend upon the magnitude of the problem at issue, the uncertainty of the solution, the amount of money involved, the possibility of difficulties in the execution of the project, and, perhaps above all, upon the knowledge of the purchaser or his engineering staff of the class of work contemplated. Self-sufficiency is a dangerous trait in handling new engineering problems; if there exists any doubt as to the proper machinery or supplies for an important piece of work the safest course is to put the problem in the hands of a consulting engineer of acknowledged standing.

The troubles which follow undue reliance upon purely "home talent" or upon salesmen in the purchase of engineering equipment are well set forth in a contribution to the Engineering Supplement of the London "Times" of Aug. 20. The writer deprecates falling back upon the advisory services of sales engineers, discusses the preparation of specifications by amateurs, and, after exposing the pitfalls of guarantees inadequately demanded by novices or unsuspiciously swallowed whole by non-technical apparatus buyers, outlines the duties of a consulting engineer in the selection of power-plant equipment. He shows that the sales engineer, no matter how honest, cannot but be biased in favor of his concern's products. Such a man may well be called into conference in the purchase of equipment and his recommendations given due weight; but it pays to go slowly in accepting anything extensive in the way of designing from such sources.

The subject of specifications affords material for a treatise. Here the independent purchaser often fails utterly, either by loose expressions which easily can be evaded or by excessively strict requirements at unimportant points, which increase the cost of his apparatus far beyond what it would have been had a consulting engineer been employed. As pointed out in commenting upon guarantees of performances in the issue of this journal of Aug. 10, 1912, the man who buys machinery without adequate knowledge as a guide runs grave risk of getting little for his money. Attempts to cover one's own ignorance by ready-made specifications based largely on the generalities of manufacturers' tenders are pretty certain to result disastrously. Makers' specifications seldom include everything necessary for complete plants, the consequence being that the unsuspecting purchaser is finally overburdened with the cost of extras which he did not anticipate when he placed his order.

Without the consulting engineer, or, in his absence, the properly qualified technical adviser or group of experts on the staff of the purchasing organization the buyer is liable to be exposed to the evils of "leaky" guarantees, unskilled analyses of bids, scattered responsibility for the operating results of diverse apparatus assembled under a single roof, insufficient inspection of work in progress in the manufacturer's shops, delays in placing orders and in stimulating shop production, drawings lacking necessary details for the most rapid filling of orders, unreasonable or inadequate specifications, and ignorance as to the apparatus which in the long run will prove the best, all factors considered, and consequently the least costly for the required service. The cost of expert advice, in cases where the resources of one's own staff are insufficient—as they often must be on account of rapid progress in every branch of engineering—is too small a proportion of the total expense of a well-conducted undertaking to justify the slightest hesitation on the part of a wise purchaser to avail himself of this aid toward economy.

The Closed-Shop Idea in Engineering

After a few more fiascos relative to the chief engineership of the Chicago Board of Education perhaps outside engineers will take the hint and cease spending good money and valuable time taking examinations for this $6,000 position. The municipal Civil Service Commission has made a valiant effort to fill the position since 1909 by interspersing promotional examinations between those open to non-residents, some of whom have been certified, but for one reason or another have not secured the position. There is one exception. The late William H. Bryan, of St. Louis, held the office for a week before he died. But in an interview with a representative of this journal he intimated that matters were going to be made extremely unpleasant for him.

Early in September practically all those duties requiring the skill of a high-grade designing expert were transferred in a quiet way to the school architect, in whom, perhaps, they should have been vested in the first place; but the salary remains at $6,000 annually. Several days after the order was passed a score of engineers, some from other cities, but all presumably unaware that the duties of the coveted position had been reduced to the supervision of maintenance and operation of the mechanical appliances and of the janitor service, proceeded to battle for several days with a strenuous examination qualifying them to direct the design, construction and operation of sanitary and electrical apparatus, lighting, heating, ventilation and plumbing.

It is high time that the board ceased acting in childish fashion toward engineers it proposes to employ. If it does not want a designing engineer and can afford to pay $6,000 for the supervision of the janitor service and the mechanical equipment, let the facts be known, an examination be drawn up accordingly, and the job given to the lucky aspirant. On the other hand, if the board feels that the job must be fitted to the man—a procedure entirely opposed to the promotion of efficiency—stop the nonsense, give him the position temporarily and let him keep on taking the closed-shop promotional tests until he learns the answers to the questions.

Sanitary engineers will recall an instance resembling the treatment Mr. Bryan expected to receive when ten years ago Chicago sent out a call for experts to qualify for a high-salaried sanitary office. Unluckily, an outsider was certified, but was peremptorily requested to leave town within twenty-four hours.

Provincialism crops out occasionally when an engineer travels beyond the confines of his native land. Strange tales of continued nagging and discourtesy, and even of rough treatment, are sometimes told by those who venture on the other fellow's preserve. Even in passing from State to State the millennium has not yet come and home-produced engineers are often employed, even though dry rot in organizations is thus assured.

Of a somewhat similar nature was the provincialism displayed in Richmond recently, where quite a number of engineers participated in a mock concrete-bridge competition. Too much valuable time and money are wasted in these fruitless examinations and competitions for engineers to pass them by without protest. The individual cannot accomplish much, but engineering societies, local as well as national, could effectively enter their protests against procedures of this sort.

Indictments for Bad Roads

New terrors for the holder of public office —especially those who have to do with engineering works—are disclosed in the indictment of Pennsylvania State highway officials last week on the charge of "misdemeanor in office." For the misdemeanor, it appears, consists of allowing several important highways of Schuylkill County to remain in an impassable condition. The grand jury added to its presentment the assertion that the State roads to which objection is made are "common nuisances," and with the highway commissioner were indicted his chief engineer and district superintendent, and a road inspector. The incident need not be taken too seriously, for in Pennsylvania there has been developed a wonderful ingenuity in the use of legal processes for purposes of political demonstration or revenge; and in this class of proceedings many are indicted but few are convicted, or even tried. While it is believed, therefore, that Commissioner Bigelow and his staff have little or nothing to fear from the charge brought against them, the theory upon which the grand jury has proceeded has elements of great possibilities. The theory is substantially that the State has assumed responsibility for certain roads and appointed a highway commissioner to build or maintain them. If this official does not perform the duties of his office as to any road or group of roads he becomes a misdemeanant. No inquiry was made, so far as is known, to ascertain whether or not the commissioner had sufficient State funds for carrying on the Schuylkill County and other road work. If he had the money he may not have been able to get the material or labor. These things do not concern the grand jury. It wanted good roads, and as it could not get them it proposes that the highway commissioner himself is to be put to breaking stone.

If this theory should prove tenable, it is evident that it would tend materially to reduce the visible supply of office seekers. Should a street-cleaning commissioner fail to clean all streets; should a bridge engineer fail to erect structures with all desired speed; should a board of water-supply commissioners fail to make satisfactory progress; should public building authorities not finish their work as rapidly as it might be done—in all such events indictment for misdemeanor and for maintaining a nuisance would be in order according to the rule that it is now sought to establish in Pennsylvania.

More seriously speaking, some little good may come out of this proceeding if it exhibits to the public view the foolishness of State assumption of road improvement and other widely distributed public works without the provision of sufficient funds to carry out the work to be done. For in that case even inefficient local improvement work is stopped while the State fumbles around with an undertaking too great for its resources or its enterprise. But no State has money enough to carry out all of a road improvement program at one time. Somewhere, therefore, there will be roads in bad condition. Anywhere that this happens the Schuylkill County grand jury thinks that indictment of the State highway department is the remedy to apply. More money with an efficient organization to spend it would, however, build more roads than all the indictments that could be framed.

Cost of Manufacturing Electricity in Very Large Stations

Interest in the cost of manufacturing electricity continues to grow in engineering circles. The subject is unquestionably important and affords a fertile field for discussion. A rather philosophical treatment of this question by Mr. H. M. Hobart in the current issue of the "General Electric Review" deserves consideration in view of its relationship to the paper of Messrs. Scott and Gorsuch, referred to editorially in this journal on May 24, and also on account of its suggestive analysis of present costs in stations of very large size. The author's connection with many large power investigations lends much interest to the hypothetical figures carried through the article, while the logical method of handling the latter is decidedly worth adopting in the analysis of operations in specific plants.

Mr. Hobart approaches the economics of power production through the fuel calorimeter, pointing out the relations between the thermal characteristics of coal and oil and the yield in kilowatt-hours at the busbar for a given maximum station efficiency. By including each step in the calculations he shows how easily one may work from the standard of kilowatt-hours per pound of fuel to the cost of coal or oil per unit for energy delivered, given the thermal efficiency of the installation and the price of fuel in bulk or per unit of weight. As the overall efficiency of large steam-turbine stations capable of generating 100 million or more kilowatt-hours yearly averages about 11 per cent, and as that of an equivalent internal-combustion-engine plant runs around 22 per cent the year through, it is but a few steps to the determination of minimum fuel costs for a stated output and a specified price averaged throughout a given period, assuming that the best management is rendered each installation. In other words, by assuming values of station efficiency representing the best average practice from one year's end to another, the author works out some very interesting conclusions regarding the economic possibilities of large steam-turbine and internal-combustion plants, and in so doing shows the reader the ease with which similar problems can be solved for other conditions by this method of connecting thermal characteristics and energy output. And this is not only done with production costs in mind, but, in addition, the author considers investment and administration costs along the lines of accounting broadly favored in the Scott-Gorsuch paper, finally studying the influence of power factor and load factor on costs and the principal factors affecting rates for electrical energy.

Upon the assumptions made (11 per cent and 22 per cent thermal efficiency for the turbine and the internal-combustion plants respectively) the paper shows that the fuel cost per kilowatt-hour in a plant capable of generating 200 million units per year will be about 0.48 cent with either installation if coal of 14,500 B.t.u. (4.25 kw-hr.) per pound costs $4.50 per short ton and crude petroleum costs 4 cents per gallon. This shows the futility of mere efficiency to reduce costs, since the first cost of the fuel may be so great in one case that it may far more than offset its more economical use in a given prime mover; while, on the other hand, low fuel costs may in some instances be obtained only at the expense of such heavy investment costs that the latter are prohibitive. All this is effectively elaborated in the paper, with the final result that for a station of the foregoing capacity the author finds that the cost per kilowatt-hour may be figured at 0.67 cent for production, 0.20 cent for investment, 0.05 cent for administration, or 0.92 cent in all for the steam-turbine plant and at 0.73 cent for production, 0.26 cent for investment, 0.05 cent for administrative and 1.04 cents for the total charge for the internal-combustion installation. Incidentally, Mr. Hobart points out that with coal costing only $2.25 per ton, and in many central stations good coal is obtained at this price, then the oil would have cost four times as much as the coal per kilowatt-hour of calorific value, and even could the oil engine station have had 44 per cent overall efficiency, the fuel cost per unit of output would have been as great as in the 11 per cent efficiency steam-turbine station. In determining the investment costs, the author stated that $60 per kilowatt of capacity is a reasonable average cost for installing a 60,000-kw steam-turbine plant, such as would be required for the service here discussed with a 50 per cent load factor, compared with an investment cost of $80 per kilowatt in an equivalent oil-engine station. Interest was taken at 5 per cent, taxes at 1.5 per cent, insurance at 1.5 per cent, and 3 per cent was allowed for amortization on the basis of twenty years of equivalent life for the entire investment in each case. It must be admitted that few if any data are generally known which will as yet enable one to predict the comparative useful lives of very large generating plants, whether using steam turbines, oil or gas engines; but the author's assumptions in this regard do not materially affect his conclusions.

Little space need be taken to review the conclusions which the author reaches regarding the desirability of a high load factor or his interesting discussion of the influence of power factor on cost, applying in the latter the researches of Professor Arno. The benefits of a high coal factor, both on the side of production economy and reduced investment, have often been commented upon in these columns. As for the power factor, it is interesting to note that where this is less than unity, and with leading current, there will be for slight departures from unity but little increase in the cost of manufacturing electricity, since the extra capacity required for the greater current will be partly offset by the decreased generator excitation required. But as the power factor, with lagging current, falls below 80 per cent, and especially with the load factors only too common at present even in good-sized stations, the cost of manufacture rises very rapidly. As for the distribution system and rates thereon, the results of calculations in a previous paper by the author quoted in the current article, combined with passing comments upon the elements affecting service costs per consumer, are of much practical interest, and their future elaboration with examples drawn from industrial power applications would add not a little to the valuable matter thus far published on power economics.

Downstream View of Diversion Dam, Canal Intake at the Left

Puntledge Hydroelectric Power Plant—Part I

An Ultimate Development of 19,000 Horsepower which Will Supersede Separate Steam Plants Used for the Operation of Coal Mines at Vancouver Island

Canadian Collieries (Dunsmuir), Ltd., owns and operates a group of coal mines in the vicinity of Union Bay, Vancouver Island, B. C. The power for the operation of these coal mines has been generated heretofore by separate steam plants located at the various mines. About three years ago the problem of renovating and expanding some of these steam plants presented itself for solution.

General estimates were made of the capital cost of such improvements and also of a central steam-electric power station with a distribution system to the respective mines. At the same time surveys were made to determine the cost of hydroelectric power for the same purpose. Comparative estimates showed that with coal possessing a calorific value of 14,000 B.t.u. at $2.50 per ton delivered at the power station the cost of installation and operation was greatly in favor of the hydroelectric development, which method was accordingly adopted, resulting in the construction of a hydroelectric plant on the Puntledge River from specifications and detail designs prepared by the firm of H. K. Owens-A. V. Bouillon, civil and mechanical engineers, of Seattle.

GENERAL LAYOUT OF PLANT

This river is the outlet of a lake of the same name with an area of approximately 9 sq. miles and a low-water surface elevation of 420 ft. above sea level, its outlet being about 8 miles from tidewater. Puntledge River has an observed flow ranging between 330 and 3500 sec.-ft. and by building an impounding dam at the outlet of the lake to raise the water 23 ft. and cutting the bottom of the outlet to a depth of 5 ft. below low water an available storage capacity of 23 ft. of depth is obtained, which is equivalent to about 132,000 acre-ft. Considering with this the regimen of the river as recorded during the short period of observed flow, it is estimated that a constant flow of at least 800 sec.-ft. can be maintained for use in the development of power. The rated capacity of the ultimate plant will be about 19,000 hp at the turbine shaft. This being in excess of the requirements for the owner's own industries, the spare power is intended to supply other industries either established or contemplated in that vicinity.

From the impounding dam the water flows in the natural bed of the river to an intake reservoir formed in the bed of the river by a diversion dam about 2½ miles below the lake. This dam consists of a spillway and an intake with head gates, which divert the water for power development into a canal and flume line 3400 ft. long, the latter discharging into a forebay forming the intake of the penstock system.

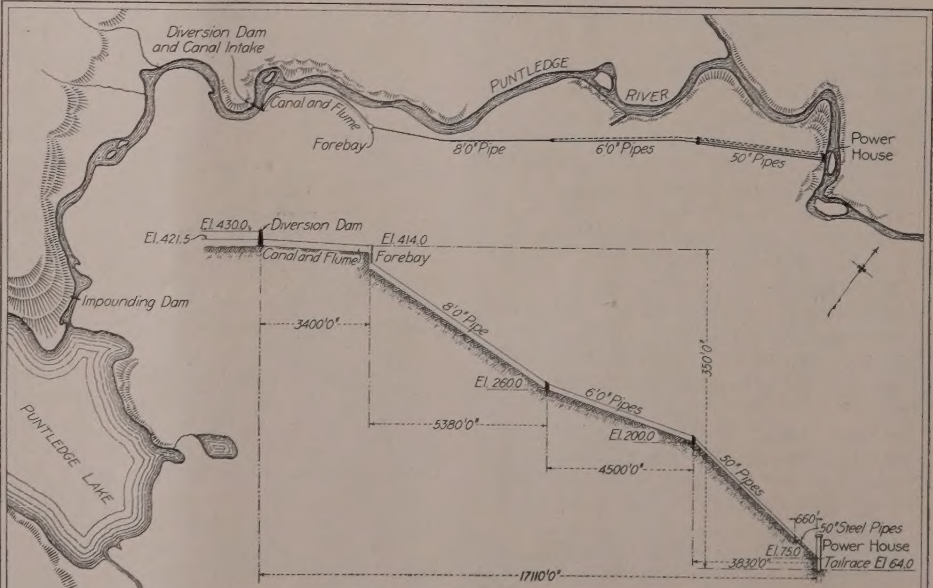
From the forebay the water is carried in a wood stave pipe 8 ft. in diameter and 5380 ft. long to a Y structure with two outlets for pipes 6 ft. in diameter fitted with gate valves. Only one-half of the ultimate penstock system is now built from this Y to the power house, and the next section of the present system consists of one line of 6-ft. stave pipe, which is to be duplicated at some future time. This pipe is 4500 ft. long and leads to a junction structure having inlets for the two 6-ft. pipes and outlets for four 50-in. penstocks, the interior of the structure forming a chamber common to all connected pipes. Gate valves are installed here at both the inlets and the outlets. There are to be four penstocks in the completed system, but only two are constructed for present purposes. These 50-in. pipes are built of wood staves and steel bands for a distance of 3170 ft.; for the remaining distance of 660 ft. to the power house the pipes are built of steel. Only one-half of the projected generating machinery has been installed, consisting of two reaction-type turbines with horizontal shafts direct-connected to generators of the alternating-current, revolving-field, twenty-five-cycle, three-phase type.

The plant differs but little in general outline from the usual scheme of any similar hydroelectric development, but in detail there are some features which are of unusual design.

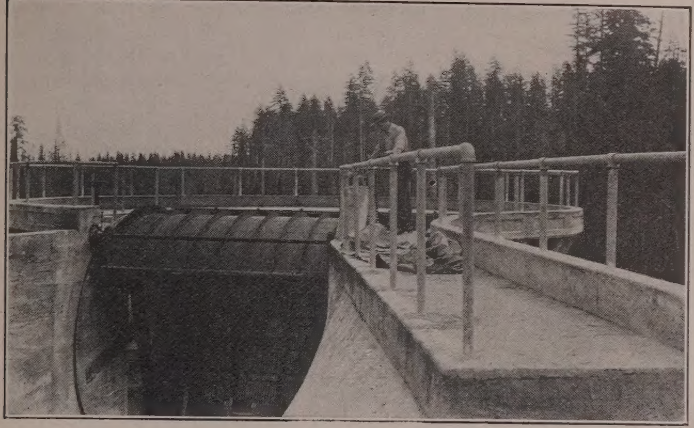
IMPOUNDING DAM

The impounding dam is approximately 300 ft. long, with a crest elevation of 445 ft. and a gate sill elevation of 416 ft. The gateways in this dam, together with spillways aggregating 100 ft. long, give a capacity of flood discharge of 10,000 sec.-ft., which is in excess of the highest recorded outflow from the watershed. In general type the structure is a buttressed wall of concrete partly reinforced. A special feature of the dam is a log sluice required by the Government regulations. This was accomplished by omitting the wall in one of the panels and substituting stoplogs laid one over the other and readily removable one at a time to obtain the necessary depth for the passage of logs at any stage of water elevation. The storage of water and its flow over the spillways are controlled by wooden needles closing the spillway wickets, and are so made and installed as to be readily removed in time of flood. These needles are made of 3 x 4 in. lumber, 12 ft. long, dressed to 2⅝ x 3⅝ in. and fitted with forged ends for protection and handling.

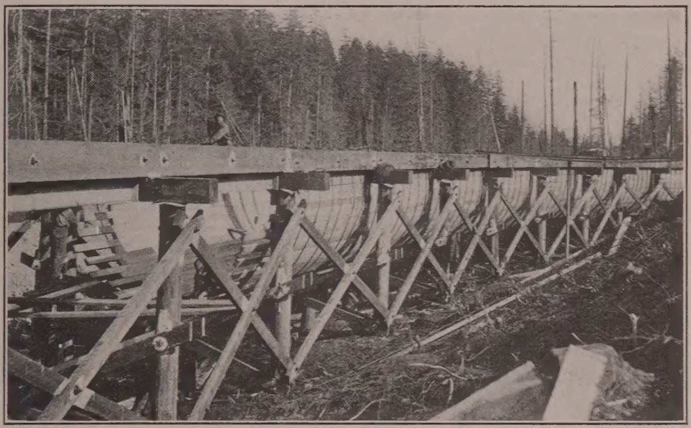
There are six gateways, each 5 x 6 ft., controlled by steel gates of the butterfly type with vertical axes. The gates are stepped in bearings, set in the sill and the spindle extends to the gallery at the top of dam and through a cast-iron pedestal frame which carries the weight of the gate and spindle and a



Plan and Profile of the Puntledge River Hydroelectric Power Plant



Forebay with Rack-Operated Tainter Gate



Wooden Flume in Course of Construction

worm-operating gear fitted with double hand cranks. The gates fit loosely into the sluiceways, as there is no intention to obtain watertightness, and their vertical position is adjustable at the overhead thrust bearing.

The dam is built on solid rock. The concrete is composed of ordinary aggregates in the proportion of 1 Portland cement, 2½ sand and 5 gravel, with approximately 10 per cent of hydrated lime added to the cement as a neutral filler to obtain watertightness. To insure against uplift of the structure from pressure in horizontal seams, which might exist in the underlying rock, a number of weep holes were drilled in the rock and were fitted with pipes leading up and out to the downstream face of the dam.

Diversion Dam

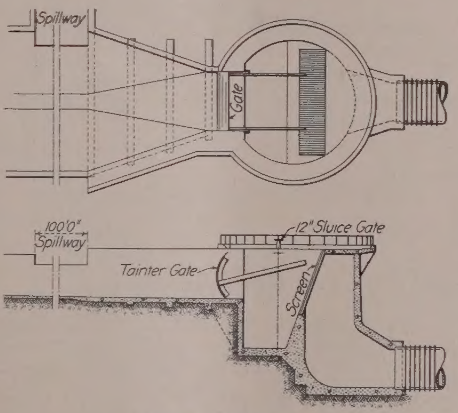
The diversion dam is also of concrete, partly reinforced at the intake end. The spillway is 100 ft. long. As the Government regulations required a fish ladder, one was built in one of the concrete abutments so as to be protected from damage in time of flood. There are two 6 x 7 ft. gateways in this dam. The gates are of steel plates riveted on steel channel frames and are fitted with rollers. Watertightness is obtained by flexible diaphragm strips bearing upon the face of the gateway along the sides and top and by a cushion on the bottom, which is seated on a steel channel sill laid inverted in the concrete floor. The gateways are protected by sectional steel screens designed for convenient handling and upkeep. The lifting gears for the gates are of the rising screw-stem type, the stems being 2½ in. in diameter with bevel-geared operating stands and hand cranks located on an overhead working platform.

Canal and Flume

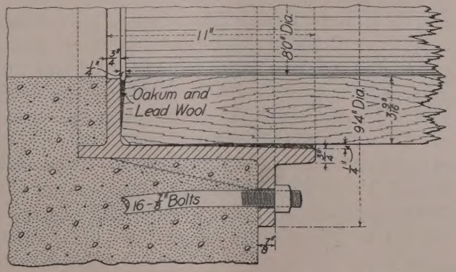
This part of the work has alternating sections of canal and flume, in which there are many sharp curves, necessitated by the broken nature of the ground. The canal sections occur in solid rock, sand, gravel and clay, all being lined with concrete except some parts cut through impervious clay. The intermittent sections of canal are connected by a wooden flume built of staves laid on steel suspension rods carried by and between side girders laid on bents placed 16 ft. center to center.

The flume is approximately semicircular in shape, 12 ft. wide and 7 ft. deep; the staves are made of 3 x 6-in. lumber, dressed to 2 9/16 x 5 7/16 in.; the suspension rods are ¾ in. and are adjustable over each girder by screw ends, with nuts over specially designed washers. These suspension rods are spaced 24 in. center to center. The girders are built up of double fir timbers 8 x 16 in., with space between fixed by separators and through which the suspension rods pass. The flume bents are built on concrete piers, except across a swamp, where the flume is carried on piles driven to solid bottom.

The canal and flume line terminates with a short section of reinforced-concrete canal, the shape of which is gradually adjusted for proper entrance into the forebay structure, and before reaching the forebay one side of the canal is formed into a spillway of capacity sufficient to carry away the entire flow to this point. A rotary balanced steel gate of the Tainter type and operated by a hand gear is installed at the canal entrance to the forebay to regulate or to shut off the supply from the canal and flume line. This gate is 12 ft. wide and approximately 10 ft. high. It consists of two main arms made of steel channels gusseted to the gate frame. The gate is balanced by a cast-iron beam of heavy section, fitted as a spreader at the end of the supporting arms which extend from the trunnion bearings within the forebay for this purpose. Watertightness of this gate is obtained by flexible diaphragm strips fastened to the sides of the gate and sliding in grooves formed in the side walls of the canal. The seats for the strips are brass bars laid in the grooves and secured in the concrete. The bottom of the gate is fitted with a soft cushion which seats on the concrete floor of the canal; and while it was not intended to have this gate absolutely watertight it is found to be practically tight in actual service, and it works well in other particulars.



Detail of Forebay



Wood-Pipe Junction with Forebay

Forebay

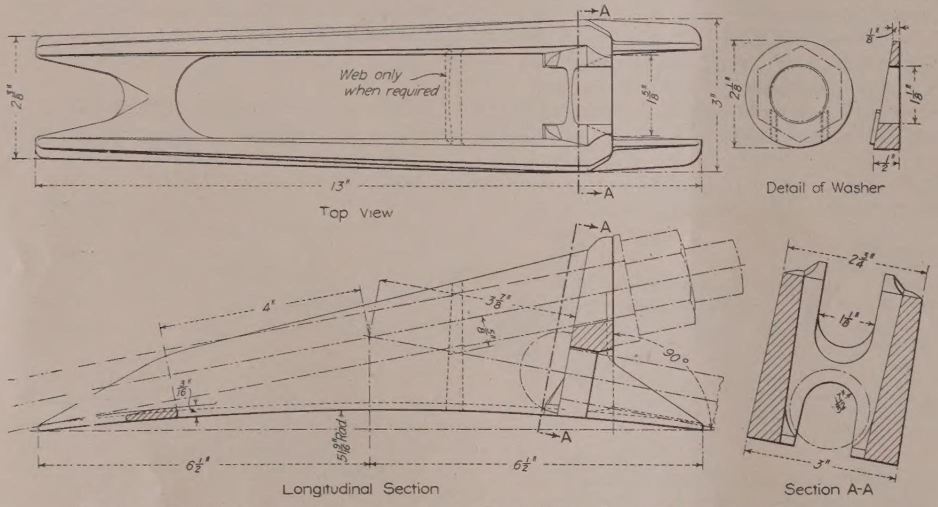
Essentially, the forebay structure is a vertical reinforced-concrete cylinder, with the canal entrance on one side and the pipe intake on the opposite side. The basin formed by this structure is divided into two sections, which are separated by a weir and screens. The section into which the canal discharges forms a sedimentation basin with floor sloping to a sluicegate; the other section forms the intake of the penstock pipe and is completely inclosed.

The concrete in this structure is composed of the same aggregates as already specified for the impounding dam, including the same proportion of hydrated lime, and it is found to be watertight in service. The outlet connection of the 8-ft. pipe is funnel shaped, 12 ft. in diameter at the inner end and tapering to 8 ft. in diameter in 7 ft. of length. For connection with the wood-stave pipe a cast-iron ring was laid and anchored in the concrete at the mouth of the outlet, the cross-section of which forms a rabbet for the pipe staves with tapering clearances which provide calking spaces all around the pipe outside and at the end of the staves inside. The calking materials used are oakum and lead wool, and the joint thus obtained is very satisfactory.

Stave Pipes and Shoes

The staves in the 8-ft. pipe are made of lumber 4 x 6 in. finished to a thickness of 3 9/16 in. and an outside width of 5 9/16 in., there being fifty-eight staves in the circle. The staves for the 6-ft. and 50-in. pipes are made of lumber 3 x 6 in. and are planed to a thickness of 2 9/16 and 2½ in. respectively and maximum widths of 5⅝ in. for the 6-ft. pipe and 5 9/16 in. for the 50-in. pipe, forty-three staves forming the full circle of the former and thirty-one staves the same in the latter. These staves are all made of Puget Sound or Douglas fir, in the specifications for which clear lumber was required and sap excluded. The stave ends are grooved for a steel tongue ⅛ in. thick and 2 in. wide, which provides watertightness at the stave joints. The bands as adopted for all pipes are uniformly 1 in. in diameter, the maximum spacing being 12 in. center to center; but this spacing occurs at the upper end of the 8-ft. pipe only; the minimum spacing is 2 in. center to center, which is as close as bands and shoes of the size adopted can be laid, and where the load is reached requiring spacing below said minimum the size of the piping is changed. It was this feature of the design which determined the lengths of pipes of the respective sizes in the penstock system, according to the brief description first given.

The bands are made in two sections, one having threaded ends and the other button-head ends. The threads are rolled on. Each band has two coupling shoes of malleable iron. A great deal of study was given to the design of the shoes or couplings for pipe bands and a number of standard shoes offered by manufacturers were examined, and some were tested with a view to their adoption, but none of the ready-made shoes offered was found satisfactory, it being apparent that the types of the larger sizes of shoes offered were mere expansions of original designs of shoes for bands and pipes of comparatively small sizes, without adequate provision or increase in scantlings having been made to balance the increases in sizes of either pipes or bands. The shoes used on the various pipes of this system were therefore designed especially to meet the actual requirements. In testing some of the ready-made shoes offered it was found that the threaded end with nut and washer would easily slip off when under stress, even before any noticeable deformation of the shoe would take place, and some of them had such a narrow margin of safety in this particular that they would not be safe to use on pipes of any considerable size. Careful attention was therefore given to this feature of the design of the shoes.



Design of Shoe for Steel Band on the 96-Inch Pipe, Puntledge Power Plant

On the shoes for 50-in. pipe a declivity as well as a dish is provided in the face of the lugs to securely hold the band end; this was considered safe for the pipe of this size and curvature of band, but for the pipes of the larger sizes, the curvature of the bands being less and the tangent end longer, it was deemed necessary to provide more declivity in the face of the lugs and to use tapered cast washers, which lock the ends in place. Shoes of each size were made from preliminary patterns, and tested and guided by the results of these tests the patterns were modified slightly as deemed advisable until shoes were obtained in which the strength developed equalled the average elastic limit of the bands. The weights of the respective shoes was held, at the same time, to the minimum commensurate with their strength as thus determined. In other words, the shoes of the three sizes as designed for these pipe lines are proportioned to withstand without any deformation the load which the band is capable of carrying to its elastic limit.

(To be concluded)

Electrification of Steam Railroads in Mountain Districts

During the last ten years the Southern Pacific Company has been investigating the question of electrification of its three outlets from the central valleys of California. The earlier reports were favorable to electrification, but when the company went independently and more exhaustively into the investigation the officers reached a unanimous conclusion adverse to electrification. Figures and practical reasons supporting such a conclusion, as applied especially to the line through Tehachapi Pass, are given by Mr. Allen H. Babcock, electrical engineer for the company, in a paper presented at the two hundred and eighty-fifth meeting of the American Institute of Electrical Engineers at Vancouver, B. C., Sept. 9-11, 1913. The paper is abstracted below.

The section under consideration is about 68 miles long, from Bakersfield east to Mojave. From Bakersfield to Summit, 49.5 miles, there is a vertical rise of 3764 ft., the average grade being 1.44 per cent. The average curvature is equal to a constant 3-deg. curve. From Summit to Mojave, 18.3 miles, the vertical descent is 1285 ft., the average grade being 1.33 per cent and the average curvature 0.79 deg. On each slope the ruling grades, since curvature is not compensated, is in effect 2.4 per cent, the maximum grades being long enough to fix the weight and power of the locomotives.

TRAFFIC CONDITIONS

In determining the energy consumption of trains moving over the mountain the actual characteristics of the line were used, but in determining load diagrams and substation spacings and capacity close approximations were made to take care of the ruling grade, curves, etc.

The average freight train, eastbound, weighs 2000 tons, exclusive of locomotives. Four consolidation-type locomotives, or their equivalent in Mallet compounds, are used to haul this train from Bakersfield to Summit. From Summit to Mojave one locomotive is used and the other three return deadhead to Bakersfield. The westbound freight trains, because much of the western movement consists of empty cars, weigh normally 1250 tons. They are hauled by three consolidated locomotives, or their equivalent in Mallets; the helper engines cut out at Summit and return light to Mojave.

In order to provide a flexible unit is was proposed to use an electric locomotive capable of handling a train unit of 500 tons, as many per train to be used as the weight of the train would require. The weight of the electric locomotives is assumed at 100 tons. Passenger-train weights vary from 250 to 600 tons, for which a single passenger locomotive weighing 150 tons was provided. A maximum freight-train movement over the mountain recently consisted of twelve full-size freight trains eastbound and eight full-size freight trains westbound, in addition to the normal passenger movement, which is seven regular trains each way per day, with occasional extras and second sections.

It may be said generally that at least half the track is laid in conditions where any overhead contact system would require very expensive steel pole or bridge construction, in addition to which there are eighteen tunnels, many on 10-deg. curves, and in none of which the vertical clearance is more than 18½ ft. In a preliminary study it matters little what particular system of propulsion is chosen, upon which to base the estimates, and so for the purposes of this discussion a 2400-volt continuous-current, third-rail contact system was selected for the main line, with an overhead contact system in yards and terminals, at Kern, Bakersfield and Mojave.

FIRST AND OPERATING COSTS

In Table 1 the first costs are based on the present traffic as shown by the train dispatcher's sheets. In Table 2 the annual operating costs are taken from the reports of the fiscal year ending June 30, 1912, for steam operation, while the same traffic and reports are used, as far as they apply, in estimating the costs for electrical operation.

TABLE 1—ESTIMATED FIRST COSTS

Six 2000-kw substations, approximate spacing 12 miles, at $35 per kw	$1,610,000
Generating station; three 18,000-kw turbines, 15,000 hp oil-fired boilers, condensers, piping, building, etc.	1,760,000
Twin-circuit 60,000-volt transmission line, at $6,088 per mile, power house 20 miles from railroad; total length 70.5 miles	430,050
Overhead contact system in yards	155,250
Contact system for main line; 140-lb. contact rail on long wood ties at $10,225 per mile, 65.8 miles; siding with 75-lb. third-rail at $7,000 per mile, 21.6 miles; total	825,000
Bonding, main line, 65.8 miles at $1,120; sidings, 21.6 miles at $900; total	122;300
Block signals; required on account of change from present d. c. signal apparatus to the a. c. apparatus necessitated by using track rails for propulsion current return	175,000
Shops and inspection shed	10,000
Electric locomotives; forty-seven 100-ton, for freight, at $35,000; eleven 150-ton, for passenger, at $40,000, allowing eight freight and three passenger for shopping; total	2,085,000
Total	$7,172,600
Credit by release of thirteen passenger, forty-seven consolidation and thirteen Mallet steam locomotives, plus 20 per cent for shopping	1,464,900
Net first cost	$5,707,700

In Table 2 no account is taken of taxes and depreciation, both of which must be paid to the extent of at least 5 per cent of the net investment, which increases the net loss by approximately $285,000 per year.

CONSIDERATION OF PURCHASED POWER

With purchased power the total investment will be diminished by the costs of 20 miles of transmission line and of the generating station, it being assumed that power will be delivered at some one point on the right-of-way, whereas local conditions located the steam station 20 miles off the right-of-way. These reductions will be respectively $120,000 and $1,760,000, lowering the total first cost to $3,827,700. Table 3 gives the corresponding annual operating expenses. The difference, $236,423, should be decreased by $191,385, the approximate tax and depreciation rate of 5 per cent on the net investment of $3,827,700, and there is left the wholly inadequate sum of $45,038 with which to purchase 53,000,000 kw-hr. at a load factor of about 20 per cent; with no profit to show for an investment of nearly $4,000,000.

At any time there may be four passenger and four freight trains pulling up hill simultaneously, taking a total of 32,720 kw alternating-current input to the line. This is not the maximum number of trains that is on the mountain regularly, but represents only those taking power. A slight derangement of schedules, an extra freight movement of citrus fruits or oil, or a blockade, will cause congestion beyond any possiblity of estimating. This traffic must be handled as circumstances require. It cannot be spaced conveniently for power demands, as many engineers and power men have suggested, but the terminal yards must be cleared as the cars accumulate. Under such conditions Mr. Babcock does not think any power company in the west-coast country would care to take such a load for any such yearly return as would be possible, even ignoring the matter of depreciation and assuming the whole difference of $236,423 to be available for the purchase of power.

TABLE 2—ANNUAL OPERATING EXPENSES

	Steam	Electric
Substation labor and supplies........		$59,700
Power house labor at $64,780; repairs and supplies at $20,000; total......		84,780
Transmission and contact system maintenance; high-tension transmission at $50 per mile, overhead construction at $250, contact rail at $150 for main line and $100 for sidings, main-line bonding at 10 per cent of cost and side-track bonding at 5 per cent; total		36,576
Maintenance of way as affected by locomotives; estimated	$126,890	83,285
Locomotive repairs; figures for steam taken from records and for electric assumed at 4 cents per mile........	270,990	70,701
Passenger enginemen's wages; for freight service assumed to be the same	48,300	29,100
Fuel; oil at 40 cents per barrel; cost for steam taken from records; cost for electric estimated from gross average daily passenger tonnage of 4050 and freight traffic of 16,040 tons east and 10,600 tons west plus 10 per cent for slow movements, giving 144,000 kw-hr. per day at station: 70 per cent over-all efficiency of transmission	240,852	100,530
Total annual cost of operation...	$687,032	$464,672
Bond interest on cost of electrification at 4½ per cent...............		256,847
Net totals exclusive of depreciation.	$687,032	$721,519

TABLE 3—ANNUAL OPERATING COSTS WITH POWER PURCHASED

	Steam	Electric
Substation labor and supplies........		$59,700
Transmission and contact systems maintenance		35,576
Maintenance of way as affected by locomotives	$126,890	83,285
Locomotive repairs	270,990	70,701
Locomotive engimen (passenger).....	48.300	29,100
Fuel	240,852	
	$687,032	$278.362
Bond interest at 4½ per cent........		172,247
Totals	$687,032	$450,609

In the face of the foregoing Mr. Babcock does not see how any recommendation in favor of electrification can be made if the opinion is based on the direct financial profit to be realized. He believes that in this case, and in the majority of cases, the profits from electrification must be realized indirectly rather than directly, considering such matters as increased track capacity, or the postponing of double-tracking.

PANAMA CANAL VISITORS during 1910, 1911, 1912 and the first half of 1913 totalled 68,024. Of these, 12,316 came in 1910, 15,790 in 1911, 20,964 in 1912 and 18,972 in the first six months of 1913, indicating a greater rate of increase during the current year than in any other period.

Appendix to Royal Commission's Eighth Sewage Report

The eighth report of the Royal Commission on Sewage Disposal of Great Britain, of which an abstract was published in the Engineering Record of Dec. 28, 1912, page 711, dealt with the establishment of some correlation between the chemical character of the water of rivers and streams and the relative purity and freedom of these latter from nuisance. The conclusion reached was that the liability of a stream to become objectionable under unfavorable conditions might fairly be estimated by the amount of dissolved oxygen absorbed in five days, when the water was maintained at 65 deg. Fahr. The percentage of ammoniacal nitrogen present was also found to be in many cases a fair index to the more or less satisfactory condition of a stream, but in the view of the commission the above mentioned oxygen-absorption test affords, on the whole, the best criterion of the liability of a stream to become offensive in warm weather. The temperature selected for the test is about the highest attained by river water in Great Britain. A chart, showing the temperature variations of seven streams during a twelvemonth, is included in the report. The lowest temperature observed was 37.2 deg. Fahr., and the highest 64.4 deg. Fahr.

The commission has recently issued an appendix to this report, setting forth in detail the experimental work which constituted the basis of its recommendations. This appendix, according to "Engineering," the source of these notes, will undoubtedly long prove a mine of valuable data for engineers concerned in the prevention and abatement of river pollution.

In commencing operations the commission addressed inquiries to all the county medical officers of England and Wales, as well as to the various river-conservation boards, and from the information thus acquired a preliminary list of fifty streams was drawn up and personally inspected. As the result of this examination, which was carried out by Mr. G. B. Kershaw, the engineer to the commission, twenty-five places were selected for further inspection and examination, but in the end attention was mainly concentrated on sixteen of the sites thus chosen. The object of the inquiry being to correlate the chemical characteristics of the waters of a stream with its state as estimated by the eye and nose, careful note was taken of the condition of its bed and banks. Stones from the shallows were examined for algæ growth, water insects and the like, and special attention was paid to any deposits of mud in the pools and banks. Fish life was also carefully noted, and at the same time data were taken of the relative rapidity of the streams, which were classified as follows:

Mean current velocity	Character of stream
From 20 ft per minute downward	Sluggish to very sluggish.
From 20 ft. to 40 ft. per minute	Slow.
From 40 ft. to 60 ft. per minute	Moderate.
From 60 ft. to 80 ft. per minute	Rapid.
Over 100 feet per minute.	Very rapid.

Based on these observations, the commission has been able to tabulate the features given in the table as characteristic of the relative purity of natural waterways in normal summer conditions.

The cases of pollution inquired into by the commission covered every condition, from insignificant to distinctly bad. The relative volume of effluent to the stream in which it was discharged was in some instances as low as 1 to 2, and in others less than 1 to 1000, but in all cases a pretty close connection was established between the results of the oxygen-absorption test and the place which would be assigned to the stream in the table, as the result of an inspection by eye and nose. The gray growth mentioned in the last column of the table appears to be eminently characteristic of an undesirable degree of pollution.

DISSOLVED OXYGEN

The efficacy of the test recommended depends upon the fact that clean river water in normal condition, and at 60 deg. Fahr., contains in solution 7 cu. cm. of oxygen, or 1 part by weight in 100,000 parts of water. If polluted, this dissolved oxygen, as the result of bacterial action, enters into combination with the organic matter present, with a consequent reduction in the amount of that freely dissolved, and this loss of oxygen thus serves to indicate the degree of pollution of a stream. All river water contains some organic impurity, and hence, if kept in condition in which the store of oxygen cannot readily be replaced from the air, there will, on keeping, be always some reduction in the quantity of oxygen contained in solution. With clean river water, however, this loss is small, while with badly contaminated streams the whole of the dissolved gas may disappear from the sample under test. The connection between the

FEATURES CHARACTERISTIC OF STREAMS IN NORMAL SUMMER WEATHER

Condition of water as regards	Very clean	Clean	Fairly clean	Moderate	Doubtful	Bad
Suspended matter...........	Clear	Clear	Fairly clear	Fairly clear	Slightly turbid	Turbid
Opalescence....	Bright	Bright	Slightly opalescent	Distinctly opalescent	Opalescent	Opalescent
Smell on being shaken in bottle...........	Odorless	Faint earthy smell	Pronounced earthy smell	Earthy wormy smell	Strong earthy wormy smell	Soapy, fecal, or putrid smell
Appearance in bulk..........	Limpid	....	Slightly brown and opalescent	Brown and opalescent	Black looking	Brown or black and soapy looking
Delicate fish....	May be plentiful	Scarce	Probably absent	Absent	Absent	Absent
Coarse fish.....	....	Plentiful	Plentiful	Present	Scarce	Do.
Stones in shallows..........	Clean and bare	Clean	Lightly coated with brown fluffy deposit	Coated with brown or dark green layer	Coated with brown fluffy deposit	Coated with gray growth and deposit
Stones in pools.	Do.	Covered with fine light brown deposit	Do.	Coated with brown deposit	Do.	Coated with brown or black mud
Water weeds...	Scarce	Plentiful; fronds clean except in late autumn	Plentiful; fronds brown-colored in places	Abundant and covered with deposit in pools	Plentiful and covered with fluffy deposit	Scarce
Green algæ.....	Do.	Moderate quantities in shallows	Plentiful in shallows	Plentiful	Abundant	Abundant in protected pools
Gray algæ......	....	....	....	....	Present	Plentiful
Insects, larvæ, etc...........	....	....	....	Plentiful in green algæ	Plentiful in green algæ	Abundant in green algæ

oxygen dissolved and the character of the stream is stated to be as follows:

Condition	Gram of dissolved oxygen in 5 days
Very clean	0.1
Clean	0.2
Fairly clean	0.3
Doubtful	0.5
Bad	1.0

The figures refer to the weight lost from 100,000 grams of the water under test. It will be noted in the above table that the figure given for a badly polluted stream exceeds the total amount of oxygen which water is capable of holding in solution at a temperature of 65 deg. Fahr. This, in fact, was frequently the case in the tests, which were made by mixing the sample under observation with tap water in such proportion that some 50 to 60 per cent of the total dissolved oxygen was taken up in the five days over which the test extended. This degree of dilution was found by experiment to yield the most concordant results.

A very fair correlation was, as already mentioned, also established between the condition of a stream and the ammoniacal nitrogen present, but the commissioners came to the conclusion that this test was on the whole the less satisfactory of the two, since the ammonia figure in the case of a stream receiving sewage effluents was not always as good an index of the actual condition of the stream as the dissolved oxygen test was found to be.

Influence of Sewer Outlets

A feature of the present series of researches has been the comparisons made between the condition of a stream above a sewage outlet and below it. Probably the most interesting observations made in the whole investigation are those on the River Avon, from Rugby to Stratford. This river is a winding sluggish stream, with a muddy bed, and there are many weirs on it. On it, or its tributaries, are situated the towns of Rugby, Kenilworth, Leamington, Warwick, Stratford and Evesham, the distance between successive centers of pollution being generally about 10 miles. For many years the stream received the untreated sewage from all the centers above named; but as matters stand to-day it would appear that the effluents sent into the stream are in the main fairly satisfactory, and that the river is maintained in a passable condition. Between Rugby and Stratford the proportion of sewage derived water in the river never falls below 1 in 8 or 1 in 9, and in one reach amounts to as much as 1 in 2.5.

The town of Rugby has a population of 21,000, and is sewered on the partially separate system. The sewage, which is domestic in character, is treated in septic tanks, the effluent from which is passed through percolating filters and irrigated over land. This effluent is stated to be bright and clear, with a clean, healthy smell, and is non-putrescible on incubation. It contained in 100,000 parts about 1.5 parts of matter in suspension, and when maintained at 65 deg. Fahr. for five days took up 1.97 parts of oxygen per 100,000.

The ratio of the sewage effluent to the dry-weather flow of the river is about 1 to 8; the river flow being 4,500,000 imp. gal. per twenty-four hours. No smell was observed near the Rugby outfall, and there was very little of the gray growth, which, as already stated, seems to be characteristic of polluted waters. Small coarse fish were present in the water below the outfalls, although in dry weather the admixture of the effluent caused considerable deoxygenation of the water in the sluggish reaches. The Coventry sewage reaches the river 13 to 14 miles further down. At this point the river has practically completely recovered from the effects of the Rugby discharge, at least so far as its chemical constitution is concerned, the chlorine content of the water alone being permanently increased. Coventry has a population of 90,000 and the sewage discharged amounted to 4,000,000 gal. per day in 1911. It is of domestic character, but with trade wastes. The treatment to which it is subjected before discharge is settlement followed by land irrigation and land filtration; the area assigned for this purpose was 378 acres at the time of the observations, but has since been largely augmented. The effluents are stated to be very variable in quality. The dry-weather flow of the river, which is said to be fairly clean on its arrival at Coventry, is put at 7,500,000 imp. gal. per hour, and the ratio of dilution less than 3 to 1.

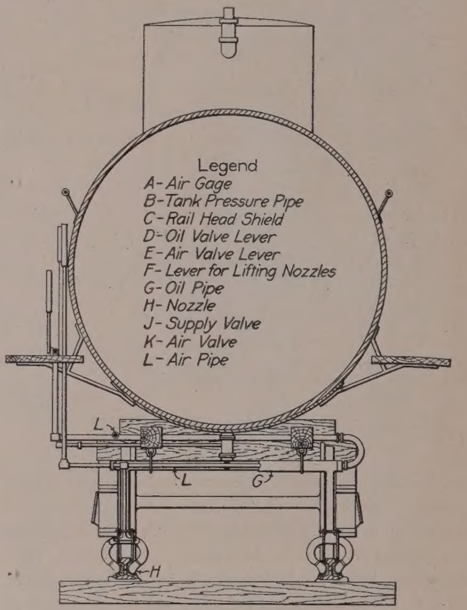
Under these conditions of an inferior effluent and little dilution there was an abundant gray growth below both outfalls, and masses of putrefying mud have formed in parts of a very sluggish reach, where many dead fish are found in warm, dry weather, accompanied by an objectionable smell. Nevertheless, the river purifies itself further on, and though it receives also the sewage effluents from Leamington, Warwick, Kenilworth, Stratford and Evesham, it is, as already stated, now maintained in a passably fair condition, so far as actual nuisance is concerned. Bacteriologically the results are much less satisfactory, although there is in all cases a well-marked progressive improvement of the river as the distance from each new outfall increases. It appears, however, that from the standpoint of bacteriology reversion to the original state of the unpolluted stream cannot in general be effected within the limits of distance which are probable in this country between independent sources of pollution.

The observations made at Berkhampstead have rather a special interest, since the sewage is discharged not into a natural stream but into the Grand Junction Canal, where practically the whole of the flow is merely that due to lockage operations. The sewage is from a population of 8,000, and is domestic in character. It is treated in closed septic tanks, followed by double-contact beds, the average flow being 500,000 gal. per 24 hours. The effluent is slightly opalescent, with a brown tint, and generally, but not always, has a clean smell. Two out of seven samples putrefied on incubation. This effluent is discharged into a pound, where the water is practically stagnant. This water had a stale, wormy smell on shaking, but there was no smell on the tow-path. Much gray fungus was present below the outfall, and the only fish observed in the pond were minnows. Several cases in which sewage is discharged direct into the stream without treatment of any kind are discussed in the report. Apparently where the dilution is sufficient this practice does not give rise to any active nuisance, but even in these cases the discharge of paper, corks and matches into the stream is objectionable, and could be avoided.

The Port of Algiers is to be extended to give an additional water area of 220 acres, according to a report to the London "Times." The Algiers Chamber of Commerce has prepared plans that will require an expenditure estimated at $19,879,000. A part of the scheme includes the construction of two drydocks, one of which is to be of sufficient size to receive the largest battleships. These improvements, it is said, will make Algiers a great factor in the protection of the French North African Empire.

Oil-Spraying Machine for Track

An outfit for spraying rail joints and fixtures with oil to prevent rust and to keep the separable parts from tightening has been in use for some time on the Delaware, Lackwanna & Western Railroad. An engine and two cars are used. The forward car is an ordinary box car, equipped with air reservoirs, air pump and piping. Coupled to this is the tank car shown in illustration. Pressure in the tank causes the oil to feed into the nozzles engaging each rail and spraying all of the separable track parts. The legend indicates the pipes, valves and other parts of the apparatus. The oil tank is charged with from 10 to 20 lb. of air. Large pipes convey the oil from tank to nozzles, and within these pipes smaller air pipes, as shown, bring either from the train line or from the air tanks a pressure of from 30 to 60 lb., as the case may require, and apply it at the nozzles, spraying the under side of the rail head, the web, base of rail, tieplates, spikes, angle bars and bolts. The spray can be regulated for any height of rail. Two of the levers along the running board control the oil and the air, and the third raises the nozzles as required to clear obstructions.



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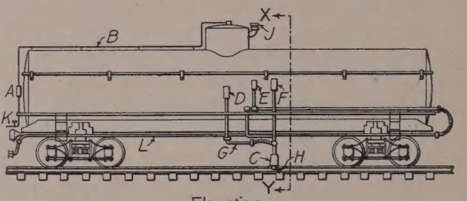


Elevation

Details of Oil-Spraying Outfit

A speed of 15 miles per hour is possible, although 50 miles per day is about the greatest distance covered as yet. In spraying 50 miles about 5000 gal. of oil are used. Two men are required to operate the apparatus, and the cost per mile, including oil, operators' wages and train service, is about $7.50.

A "Cable-Pulling" Record for the Panama Canal was made recently, according to the "Canal Record," when 17,444 ft. of cable were pulled into the ducts of the Miraflores Locks in eight hours. Of these three miles of cable 10,000 ft. were of the eight-conductor type and 7,444 ft. of the five-conductor type.

Narrows Siphon of the Catskill Aqueduct

Flexible-Joint Submarine Pipe Line, 36 Inches in Diameter and 10,000 Feet Long

By Alfred D. Flinn, M. Am. Soc. C. E., Department Engineer, Board of Water Supply, New York

In order to bring an additional supply of water from the Catskill Mountains into the five boroughs of New York City it has been necessary for the aqueduct now nearing completion to cross a number of streams and other bodies of water. At some places pressure tunnels deep in the rock have been driven and at others large steel and reinforced-concrete pipes have been used. Descriptions of these types of construction have been published in the Engineering Record at various dates. The last and in not a few respects one of the most difficult and interesting of these inverted siphons is about to be put under contract, bids for its construction being advertised for opening in the near future.

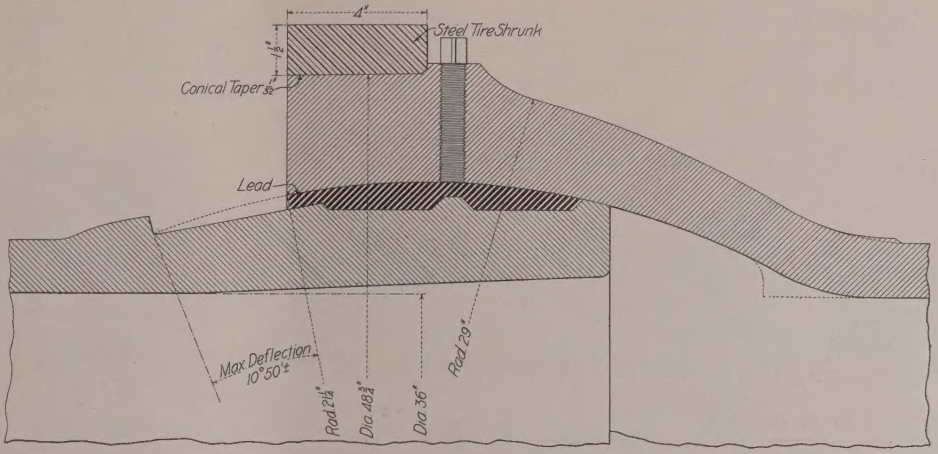
Staten Island, constituting the Borough of Richmond, the most southerly of the five boroughs, lies close to the New Jersey mainland, but legal barriers prevent a water supply being obtained from that State. Consequently Catskill water must be conveyed from Brooklyn across the Narrows of New York Harbor. This crossing is too long, the underlying rock is at too great a depth and the quantity of water to be delivered is too small to permit a pressure tunnel to be employed. After considering many schemes, a cast-iron, flexible-jointed, submarine pipe line was selected as most feasible and economical. A diameter of 36 in. was decided upon as being about the practical limit of size. On the most suitable location the line will be nearly 10,000 ft. long and will require approximately 3800 tons of pipe.

CONSTRUCTION DIFFICULTIES

Many submarine pipe lines have been laid under difficult circumstances. Types of pipe and methods of laying more or less satisfactory have been developed. The Narrows siphon will have many of the usual difficulties in an exaggerated degree and some others that are unusual. The Narrows is the main entrance to New York Harbor from the ocean. Through its channels pass the largest steamships afloat and very many other craft, making it one of the busiest places of its kind in the world. Besides the two channels the pipe line will cross an anchorage ground for merchantmen and one for men-of-war. The Government requires a minimum water depth of 45 ft. below mean low tide. Across the anchorage ground, where the depth of water is now 30 ft., dredging to a depth of about 30 ft. below the harbor bottom for a length of nearly 3000 ft. will be required.

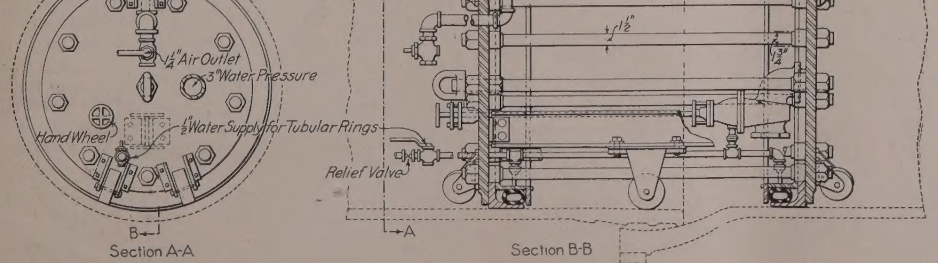
Across the channels also a trench will be dredged deep enough to permit covering the pipe at least 8 ft. with sand and other suitable material. This dredging must be done athwart the currents, rendering the trench liable to some refilling by bottom material being carried into it between the time of excavation and the laying of the pipe. The Narrows is the passage through which the Hudson River discharges. The currents commonly run at about 3 miles per hour, seaward and harborward alternately with the tides. Between tides the slack-water period is brief, and sometimes there is none. At times the flow is in opposite directions at top and bottom.

In the spring, especially after a severe winter, the river brings down large quantities of ice. At this season of year fogs are common. These conditions add greatly to the difficulties of guarding and maintaining floating equipment engaged in dredging, pipe-laying and refilling. The maximum natural depth of water is about 60 ft., but to the bottom of the trench will be nearly 72 ft. A number of soundings and wash borings were made in an attempt to get information about the profile and character of the bottom, but owing to the difficulties of the location the results are less certain than could be desired. Apparently the bottom material is mostly sand, of various degrees of fineness, with some gravel in places, overlaid with silt and sewage sludge brought down by the river. This soft deposit also varies in depth in different parts of the line. Under the circumstances described diving is difficult.



Details of Flexible Joint for Narrows Siphon Pipe

Owing to the importance and cost of this pipe line great effort is being made to attain the greatest feasible measure of permanence and security. Repairs would be exceedingly costly and tedious. To protect the pipe from damage by anchors or a foundering vessel settling upon it the 8-ft. covering was determined upon. This depth of cover resulted from experiments tried with a variety of large anchors on the location and for this purpose, also from extensive inquiries made of divers who had recovered lost anchors in the harbor. The upper portion of the refilling material will be selected with regard to resistance to anchor penetration.



Apparatus for Testing Tightness of Pipe Joints

DETAILS OF FLEXIBLE JOINT

Prolonged study was given to the design of the pipe and the metal of which it should be made. Dense, strong cast iron, as low in graphitic carbon as could be obtained, was considered the best material the market afforded. The customary 12-ft. pipe lengths are to be used, unless the pipe foundries find it practicable to increase this length somewhat. A few 6-ft. lengths may be used where change in profile is abrupt. Each length will have a flexible joint. Many types of joint were examined. The modified Ward joint, known as the Duane joint, was selected and endeavors were made to improve it. The adopted design is shown in an accompanying illustration. The principal change in the older type of joint is the addition of the gib screws to be used as pumps for forcing cold lead into the shrinkage space of the joint. The first suggestion leading to this improvement was made by a foreman in the pipe foundry in which some experiments were in progress on full-sized 36-in. pipes. His idea in turn came from an old type of screw joint for high-pressure steam pipes patented by Cornelius A. Folly, of New York, in 1891. As shown in the drawing, the 36-in. joint under hydrostatic and deflection tests gave reasonably substantial basis for the hope that it might remain satisfactorily watertight through ordinary vicissitudes of submarine pipe laying without the necessity of calking by divers after the pipe line was on the bottom of the trench, except occasional joints.

Qualities sought for the joint were watertightness, durability, strength, simplicity, quickness of makeup and the lowest cost consistent with a suitable degree of these desired qualities. Numerous types were investigated by research of literature and by experiment, and after the tentative adoption of the Duane much attention was devoted to the kind of metal to be used in the "lead space," commonly so called. The material shrinkage of soft pig lead when cast into such a space, in accordance with usual practice, was well known and the relatively shallow effect of even good calking on a joint made by pouring lead was also known; both these phenomena were further observed experimentally.

Calking even with a pneumatic hammer in the hands of a skilled workman consolidated the lead not more than ⅝ to ¾ in. from the face of the joint. When a calked joint is moved, as in the deflection of a flexible joint, the effect of the calking is almost completely nullified, as a moment's reflection will show must be the case as a corollary to the facts just stated. Lead wool was tried because it could be calked thoroughly strand by strand from bottom to face of the lead space, and being put in cold would not shrink like the poured lead. This material made joints which remained tight even under hydrostatic pressures so high that cast lead joints in the same pipe line were caused to flow until their lead was nearly out of the bell. But the lead-wool joints were so rigid that the pipes could not be deflected without the exertion of so great a force that the purpose of the flexible joint was defeated.

Method of Leading Joints

Type metal and similar alloys of lead were tried in the endeavor to find a joint filler that would not shrink in cooling, or at any rate would not shrink sufficiently to permit objectionable leakage. Nothing satisfactory was discovered along these lines. That lead would flow readily under comparatively low pressures—700 lb. per square inch—when steadily applied was demonstrated experimentally. Advantage was taken of this property of the lead when the addition of the screws to the Duane joint was suggested. The tap bolts or gib screws having been put in so that their ends were flush with the inner surface of the bell of the pipe, molten lead was poured in to fill the joint space in the usual way. After the lead had cooled, the screws were removed and a stiff mixture of graphite and thick mineral grease was forced into the shrinkage space between the lead and the bell. Cast lead plugs about ½ in. in diameter and 1 in. long were then put one at a time into each hole and forced into the joint space by screwing the bolt home, a man doing this readily by means of a 12-in. wrench. In this way about one hundred plugs were forced in through the sixteen holes.

Evidence that the shrinkage space had been pumped full of lead was given by the slight movement at the face of the joint and the oozing of the mineral grease. Such joints remained watertight before, during and after deflection in both directions and deflected easily. The surface of the lead exposed by the deflecting showed that the joint space was thoroughly filled. More rapid means than the man with the hand wrench will doubtless be devised for lead pumping in practical pipe laying. Some minor modifications of this joint have been suggested but have not been tested. The contractor, when selected after the bids are opened, may contribute some valuable ideas from his experience.

Hydrostatic Tests

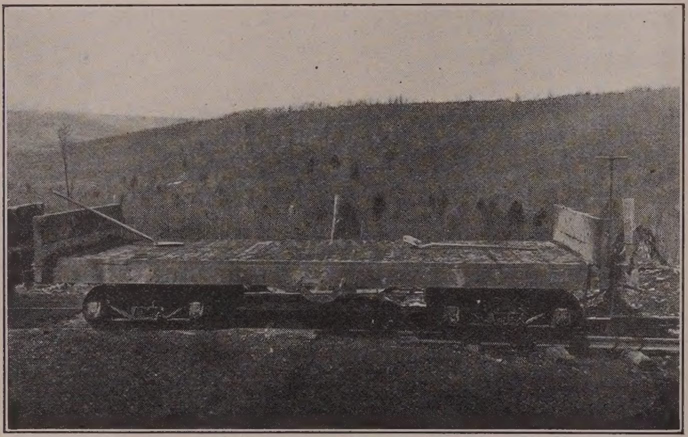
A thorough hydrostatic test of each joint above water, immediately after finishing, was also deemed desirable. Then if the joints should come up to optimistic expectations, nothing would remain to be done after the pipes sank beneath the waves. However, provision is made in the contract for testing the pipe line in successive sections during the progress of laying, so that if a few joints become defective after being submerged they can be calked by divers. Provision is made also for a preliminary test after a short stretch of line shall have been laid to determine the practicability of the method outlined. If this preliminary test should indicate that an acceptable degree of watertightness cannot be attained in this relatively simple way, resort will be had to another provision of the contract, for general submarine calking, with a guarantee by the contractor that the leakage will not exceed the rate of 40 gal. per joint per twenty-four hours. For the individual joint tests above water a special apparatus, shown in one of the illustrations, was devised and built. It was successfully used in making the full-size experiments on the various kinds of joint material. To aid in the discovery of leaky joints under water provision is made in the contract for the use of air under pressures up to 80 lb. per square inch, if necessary.

Another detail of pipe making carefully studied was the coating. It is a matter of common experience that the tar dip so generally used is not permanent. Bitumastic enamel, made in England and handled in this country by the American Bitumastic Enamels Company, was found upon investigation to have given a good account of itself through more than twenty years' use in sea water for protecting iron and steel. This material has been applied chiefly to vessels and floating docks, but has also been put on to a few pipe lines. Although much higher in first cost, its record was so good that it has been specified.

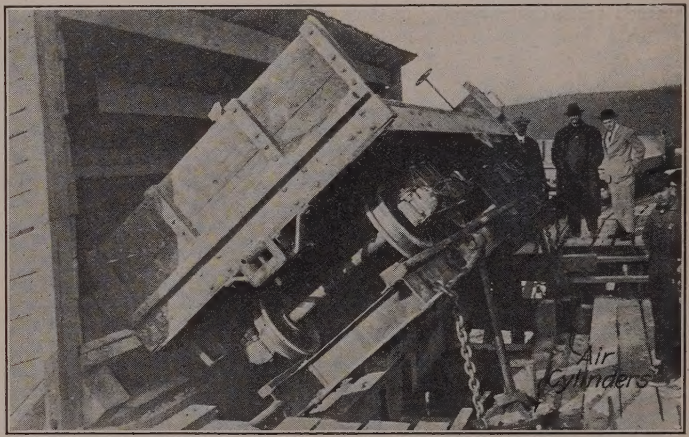
Like other portions of the Catskill water-supply system, the Narrows siphon is within the jurisdiction of the Board of Water Supply of the City of New York and will be constructed under the supervision of Mr. J. Waldo Smith, chief engineer, and Mr. Walter E. Spear, engineer of the City Aqueduct Department. In the investigations and the preparation of the contract designs and specifications the writer has been ably assisted by Mr. Thomas H. Wiggin, senior designing engineer, and Mr. Ernst F. Jonson, engineer inspector.

Low Cars and Dumping Device for Quarry Service

Two important requisites for quarry cars, as generally conceded, are (1) that the height from the ground to the top of bed should be as small as possible, particularly if cars are to be loaded by hand, and (2) that cars should be able to be dumped quickly at the crusher, in order to save time and labor in unloading. It is not possible to build a large car with dumping body without making it so high that it cannot be loaded by hand. The height is not a material objection in cars to be loaded by a steam shovel, though even in such cases a low car has some advantages. There have been several designs of cars with separate bodies hinged on one side to the underframe in connection with which an air or electric hoist engaged the side of the car opposite the crusher, lifting the body so that the load would slide into the crusher. A car constructed in this manner is expensive, as it requires practically two underframes, and if a large number of cars are used the cost of the equipment is materially increased.



Low-Platform Car for Quarry Service



Dumping Device for Quarry Service Cars

When the Warren Silica Company, of Torpedo, Pa., was installing its new quarry equipment this point was taken up with the engineers of the Orenstein-Arthur Koppel Company, of Pittsburgh, and a system was devised which would lessen the cost of the equipment and make unloading cheaper than with other types of equipment considered. The accompanying illustrations show the installed equipment. The car is very low and has double trucks. The deck is 20 ft. long, 6½ ft. wide and 2 ft. 9 in. high above the rail, thus making it possible for men to load the car with ease.

At the crusher is installed a steel frame or tipple, hinged at one side and supported on the other by the pistons of two air cylinders. The car runs on this platform, a catch engages the side of the car automatically and by admitting air into the cylinders the tipple with the car is tilted so that the car floor assumes a slope of 45 deg. while discharging the load. The cars and tipple are reported as giving satisfaction.

"Ferro-Cement" is the name given to a new road-surfacing material that is reported in the "Scientific American" as giving satisfactory service under automobile traffic in France. The material consists of a mixture of "iron straw," or threads of iron in a wiry or fibrous mass, with cement and sand. The iron is not a waste product, but is prepared by machines of special design.

Constructing a Ten-Story Concrete Building

Placing 35,000 Yards of Concrete in Floors and Columns, Bending Reinforcement Rods by Power and Handling Beam and Girder Forms

One of the largest factory buildings in the United States is now being constructed near the Pennsylvania Railroad tracks at the end of the Queensboro Bridge in Long Island City, N. Y. It is a ten-story 430 x 200-ft. concrete structure to be used by the Loose-Wiles Biscuit Company under a lease from the Degnon Realty & Terminal Improvement Company. The building will contain nearly 1,000,000 sq. ft. of floor space, and its construction involves about 35,000 yd. of reinforced concrete, 4000 tons of structural steel, 2500 tons of reinforcement bars and 80,000 bbl. of cement. Neither in the design nor in the method of construction are conventional practice or routine operations strictly followed, but special effort has been made to take advantage of the most economical selections and combinations of materials and adaptation of conditions to rapid and efficient construction.

Owing to the heavy loads imposed it was found desirable to use structural steel for the interior columns in the lower stories. The rear walls are of brick, while the remaining walls are veneered with white terra cotta. The floors are designed for loads of from 150, 200, 500 and 3000 lbs. per square foot. The basis of design for the reinforced concrete construction is the reinforced concrete regulations of the Borough of Manhattan issued Dec. 28, 1911, and now in force in all five boroughs of Greater New York.

DESIGN

The columns are supported on two hundred and fifty-six concrete piers with pile foundations to cover 80 per cent of the total area of the lot, as described in the Engineering Record of May 31, page 606. The dimensions of concrete columns required to support the very heavy floor loads in the lower stories would be so great as to produce serious obstructions and waste valuable space in the building. Structural-steel columns are therefore used up to the seventh floor, above which all columns are of reinforced concrete.

All of the exterior columns, which are substantially wall piers, are of reinforced concrete, with rectangular sections having maximum dimensions of 6 x 2½ ft., one of them about 19 ft. high in the basement story containing 13 yd. of concrete.

A large part of the seventh floor is occupied by a set of English ovens, two stories high, extending through the eighth floor. They contain more than 1,000,000 bricks, and their side walls weigh about 16 tons per linear foot. They are supported directly on plate girders riveted to the columns and inclosed in solid masses of concrete. All of the other floors and the roof have reinforced-concrete slabs, beams and girders, the latter being supported by brackets on the interior columns, which are inclosed in solid concrete with a minimum thickness of 4 in. beyond the steelwork.

Each floor is accessible through six elevator and stair shafts, the latter being provided with smoke vestibules with reinforced-concrete walls having two doors leading into the main room and one door into the shaft. The upper parts of the vestibule windows are fitted with open louvres, affording smoke ventilation in case of fire. The steel-frame windows at these shafts are glazed with wire glass, but the other windows, not being adjacent to nearby buildings, are glazed with plain glass and have wooden frames. The panels between the wall piers have very large windows, the spaces around them being filled with brickwork faced with white glazed tiles. Where wooden-floor surfaces are required the concrete slabs are covered with 7½ x 3-in. splined yellow pine planks with a wearing surface of 1⅛-in. maple boards.

CONSTRUCTION PROGRESS

The construction of the basement columns was commenced in May, soon after the completion of the foundation piers, and the first floor was completed about July 25, some delay having been occasioned by the necessity of waiting for the steel columns to be erected in advance of the concrete work. Since June 28 the construction has progressed at the rate of about four full stories in six weeks. The basement columns, in one-story lengths, and the upper sections, in three-story lengths, are erected by two steel guyed derricks, which keep them erected three stories in advance of the concrete work.

Second Floor Constructed and Columns Erected to Fourth Floor, June 28

The 1:2:4 concrete is made of Knickerbocker Portland cement and aggregate furnished by the Phoenix Sand & Gravel Company. It is unloaded from scows by a Brownhoist crane with a clamshell bucket, which delivers to 4-yd. side-dump cars, in which it is drawn by locomotives to the site.

The surface of the ground slopes from about the level of the second floor at the north end of the building to the level of the ground floor at the south end, a circumstance the advantages of which have been utilized in the installation of one of the concrete-mixing plants, which is located just outside the building about 100 ft. from the north end. The level of the industrial tracks on which materials are delivered is about 12 ft. above the ground floor, and cement in sacks is unloaded directly from the cars to the floor of a roofed cement platform, where it is stored and whence it is chuted to the charging platform of the mixer. Gravel is dumped into fourteen small bins with their floors pitched about 40 deg. to vertical sliding steel gates 30 in. above floor level. Through these gates the gravel is discharged into carts of 6 cu. ft. capacity, which transport it to the Ransome mixer, where four loads of gravel, two of sand and five bags of cement are charged for one batch. Because the sand is too wet to flow readily through bins like the gravel bins it is dumped from the cars to a platform and then shoveled into the carts serving the mixer.

The mixer is set in a pit so that the top of the charging hopper is on a level with the charging floor, which slopes slightly from the bins, thus facilitating the movement of loaded carts. The mixer discharges to a bucket elevated in an automatic Ransome hoist tower, which is extended as required to correspond with the successive stories. The buckets are automatically dumped in the usual manner to carts distributing the concrete to all parts of the building. The hoisting bucket is suspended from two lines, one of which leads directly from the top of the tower to the electric-hoisting engine, while the other passes over a sheave, also at the top of the tower, and is attached to a counterweight that nearly balances the empty bucket, reduces the danger of dropping the bucket too rapidly, and is considered to effect an important saving of wear and tear due to the use of the brake.

CONCRETE MIXING EFFICIENCY

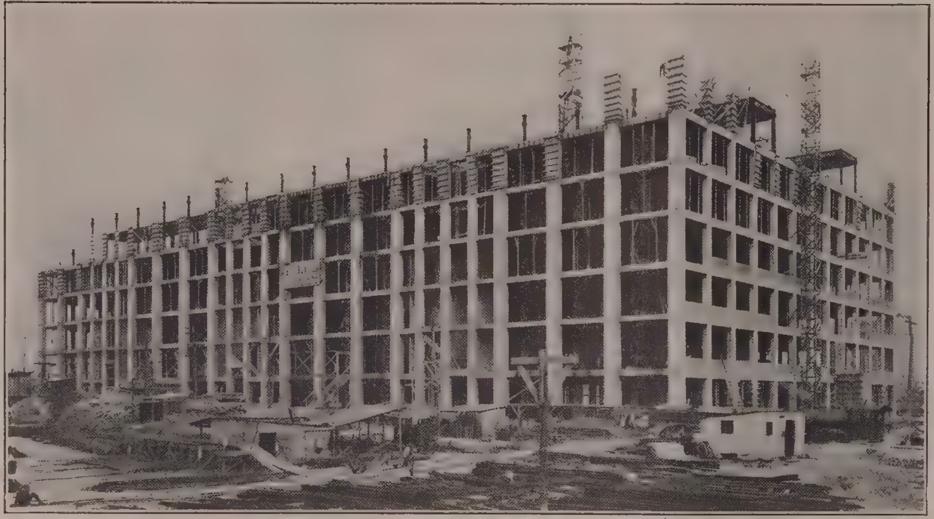
The mixing crew is composed of four men wheeling gravel, two men wheeling sand, four men shoveling sand, two men handling cement at the mixer, three men handling cement at the storage platform, two men trimming gravel in the bins, one man operating the mixer and one man operating the hoist, making a total of 19 men who have handled ten 35-cu. ft. batches of loose material in eleven minutes recorded time, and have made in eight hours a run of three hundred and forty-nine batches of 35 cu. ft. of loose materials.

Another 1-yd. mixer, located 100 ft. from the opposite end of the building and on the same side, is installed where the level of the ground is nearly the same as that of the first floor of the building and did not afford any opportunity for handling material by gravity. Sand and gravel were therefore dumped on the ground, shoveled into barrows and wheeled to the mixer, with the result that for this mixing machine of the same capacity as the one already described there was required a crew of four men wheeling gravel, two men wheeling sand, three men shoveling sand, twelve men shoveling gravel, three men handling cement at tracks, two men handling cement at the mixer, two men trucking cement, one man operating the mixer and one man operating the hoist, making a total of 30 men who handled ten 35-ft. batches of loose material in fifteen minutes and thirteen seconds, and made a run of two hundred and eighty-six batches in eight hours.

Although it was recognized that the mixing by this machine was much more expensive than by the other machine, it was not considered desirable to install a gravity apparatus, that would have been necessary to make the outputs correspond, because a careful estimate showed that the saving in expense of mixing the amount of concrete required to be handled by this plant did not quite equal the cost for installing a trestle for the delivery of materials at a greater height and the construction of the necessary bins. The best record yet made is mixing and placing 550 yd. of concrete in eight hours.

Columns

The columns have I-shape cross-sections fitted at each corner with a light vertical flange made of steel folded double with edges bent to engage the flange plates of the column. The flanges project about 3 in. beyond the structural steel, make angles of about 45 deg. with the cover plates and are connected by bent steel bars bolted to them. The doubled outer edges are slotted at intervals of about 4 in. to receive a spiral wire wound around the columns to act as anchorage for the inclosing concrete. Every interior column is inclosed by a guard made of two bent sheets of No. 18 gage steel, protecting the exterior face of the concrete to a height of 5 ft. above the floor. Special timber sills are clamped to the columns just above the finished floors to receive the erection derricks and support them clear of the concrete slabs, thus relieving the new concrete from heavy loads.



Sixth Floor Constructed and Columns Erected to Eighth Floor, Aug. 12

Forms

Forms for the floorslabs, beams, girders and columns are used about ten times each and are made of ⅞-in. boards for the floorslabs, 1¼ in. for the sides of the beams and girders and 1¾ in. for the bottoms of the beams and girders and of 4 x 4 in. for the posts. Together they required two carloads of nails. The forms for the columns are made with single panels for each side, assembled with exterior bolted yokes made of 4 x 4-in. and 6 x 4-in. timbers. All of the beams, girders and slab forms are made complete in single pieces, removed and replaced as units. They are handled by three movable towers, each of them mounted on casters.

The tower is a timber structure about 25 ft. high, pivoted below the center of gravity to a pyramidal framework fixed to the moving platform. A hand windlass is bolted to the tower below the pivot and serves as a counterweight. When not in service the tower is revolved to a horizontal position and can be moved throughout the building, with plenty of clearance from the floor above. When in service it is easily revolved to vertical position and is ready for hoisting and lowering the forms, which is easily accomplished by two men. Light wooden tripods with hand windlasses attached to them are located on the floor adjacent to the four hatchways in each story, and are used for hoisting material.

With a force of about four hundred men working one eight-hour shift daily the work is progressing rapidly. The forms are stripped when the concrete is about four days old, and the beams and girders are shored up and supported for about three weeks longer. Special provision is made in the exterior walls and wall piers for about three thousand outlets for electric lamps which will be provided to illuminate the exterior of the building.

Bending Reinforcement Steel

The concrete is reinforced by about 2500 tons of Havemeyer and twisted bars up to 1¼ in. square, all of which are bent at the site by two power-bending machines designed by Mr. R. C. Wilson, engineer for the contractor.

Stirrups and small rods are bent in sets of eight in a machine having a table provided with lever arms and adjustable pins operated by belt-driven gearing, which will make four bends in a set of rods in about ten seconds. The heavy rods, such as the moment bars in beams and girders, which are made with four oblique bends connecting the portions in the top and bottom part of the girder, are placed horizontally with their ends engaging two steel-lined slots in a rigid framework. The center of the bar engages bearings in the same plane on a car which is moved forward by a link motion driven by the same motor that operates the other bending machine, and is returned by a counterweight. This machine bends the heaviest rods singly at the rate of about one hundred and twenty pieces per hour. The rods are delivered from the bending machines to the building on a special four-wheel truck and are hoisted to the required elevation by two derrick booms mounted on a wooden tower similar to those used for the concrete elevators.

Mr. William W. Higginson was the architect for the buildings. The Degnon Contracting Company is the owner and lessor and was the contractor for the foundations. The general contractor for the building was the Turner Construction Company, of New York City. The structural steel work was fabricated and erected by Post & McCord, of New York.

Car-Float Service is beginning to be developed in the Chicago River at Chicago, and when the new lakefront harbor is in readiness the use of floats for transferring cars is expected to get added impetus.

Improvising a Pile Driver Operated by Gasoline Engine

In the Glacier National Park in Montana a 140-ft. dock was recently constructed on St. Mary's lake at a point where all equipment had to be freighted so far that it was not thought feasible to bring in a steam donkey engine for pile driving. The resident engineer was directed to use horse power for operating a 2000-lb. hammer which was already on the job. It was found, however, that a 6-hp Fairbanks-Morse gasoline engine, which had been brought in to run a bandsaw, could easily be moved to the dock site, and with this engine and a small hand derrick to begin with a mechanical pile-driving plant was devised. The mechanical advantage necessary was provided by two double blocks (using 1¼-in. manila rope), and these were operated from the drum of the derrick, driven by a belt transmission from the engine. The driver frame was built wholly of hewn timber, with the exception of the two 30-ft. leads.

The bearings of the smaller cog in the hand derrick were readjusted so that it was possible to shove it axially by means of a hand lever far enough to throw it out of gear. Thus when the hammer tripped at the top of the leads the gears were disengaged and the larger cog ran free as the hammer fell. With this equipment operation was fairly satisfactory, and the dock has been nearly completed, the average penetration of the piles being about 6 ft. into fairly soft material. For the first 60 ft. the penetration was 8 and 9 ft., and then a 4-ft. layer of large boulders was encountered, in which at least thirty piles were broken, it is said, in placing three bends. Beyond this a clay bottom was again found. The hammer can be operated, it is reported, so as to strike a blow every forty-five seconds, and in the ordinary course of the work the blows average one per minute.

The dock is a part of the development being done in the park by the Great Northern Railway. Mr. R. C. Floyd is resident engineer in charge of the work.

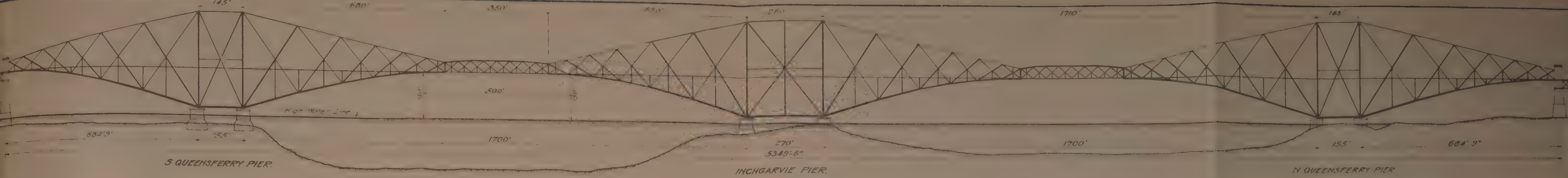
Unit Costs on Steam-Shovel Work

An example of the value of cost keeping on steam-shovel work is shown in a small contract carried out by the Aberthaw Construction Company, of Boston. About 8000 cu. yd. of dry gravel, with a small percentage of boulders, were removed from a sidehill to make room for a building foundation. A ⅔-yd. Thew steam shovel was used for the work, the spoil being loaded into dump cars pushed by hand back and forth on a standard contractors' track 500 ft. long carried on a trestle 10 ft. high. The average haul was 400 ft.

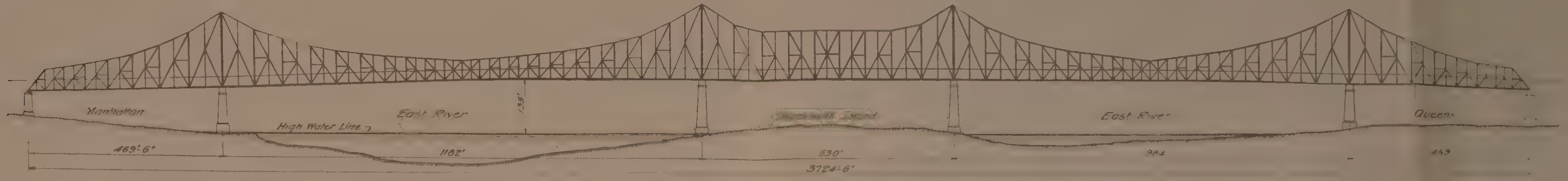
Although it is estimated by the contractor that this method saved 8 cents per cubic yard as compared with hand work, the cost was nevertheless 37½ cents per cubic yard, the item of labor alone on the shovel work proper being practically 23 cents. The itemized unit cost was as follows:

	Cents
Labor around shovel, pushing cars, unloading and spreading at the dump	22.9
Rental of shovel ($345) and repairs ($18.58)	4.5
Labor on track, including trestle	3.5
Labor for unloading, placing, etc., at the job	3.1
Rental of track	1.4
Labor at Boston end (including riggers' bill)	1.25
Freight on shovel and parts (both ways)	0.85
Total	37.5

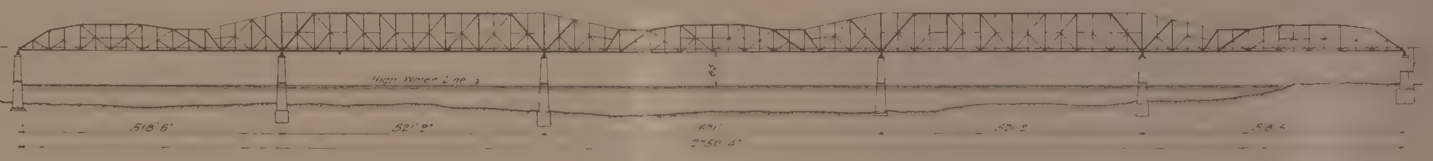
Supplement to Engineering Record, Sept. 20, 1913, accompanying article by Ralph Modjeski, on "Design of Large Bridges," published by courtesy of the Franklin Institute, Philadelphia.



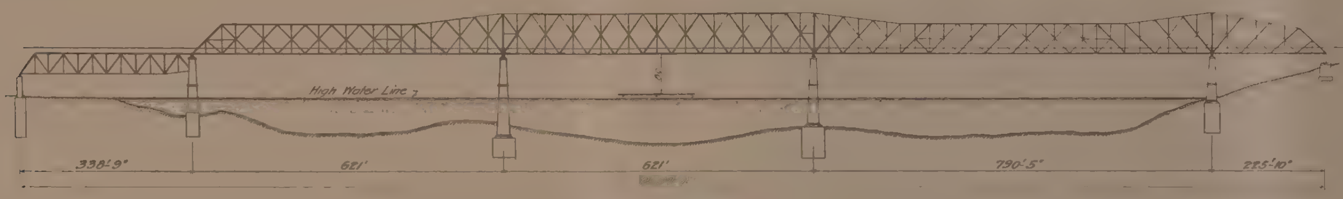
THE FORTH BRIDGE
(Near Edingburgh, Scotland.)



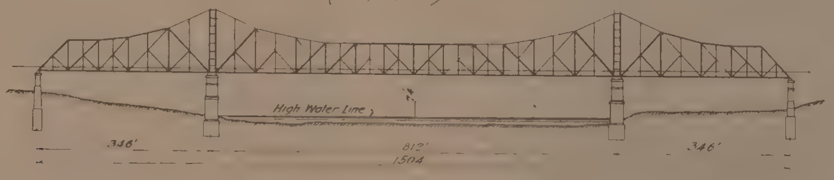
BLACKWELL'S ISLAND BRIDGE
(New York N.Y.)



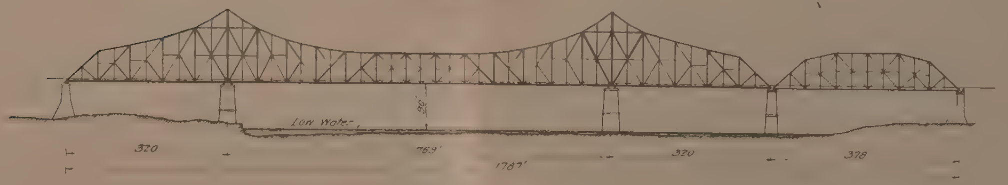
THE THEBES BRIDGE
(Thebes, Ill.)



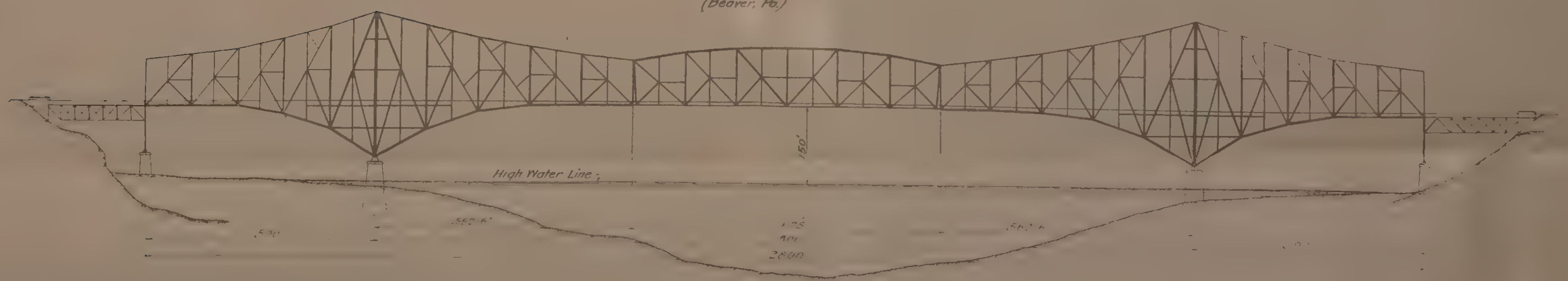
THE MEMPHIS BRIDGE
(Memphis, Tenn.)



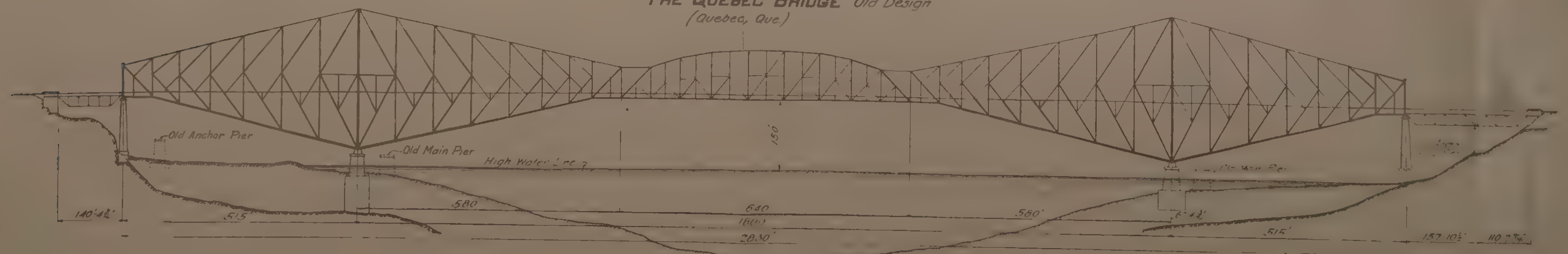
THE MONONGAHELA BRIDGE
(Pittsburgh, Pa.)



THE BEAVER BRIDGE
(Beaver, Pa.)



THE QUEBEC BRIDGE Old Design
(Quebec, Que.)



THE QUEBEC BRIDGE New Design
(Quebec, Que.)

GREAT CANTILEVER BRIDGES

SCALE IN FEET

100 50 0 100 200 300 400 500 ft

orms, which is easily accomplished by two pected to get added impetus.

Design of Large Bridges, with Special Reference to Quebec Bridge—Part I

Paper by Ralph Modjeski, Consulting Engineer, of Chicago, Presented before the Mechanical and Engineering Section of the Franklin Institute, Philadelphia

(Accompanied by folding insert of world's greatest cantilever bridges)

Some of the more important questions to be considered by the designer of large bridges and the propriety of selecting the location not only to secure a favorable site for construction but also to make the bridge most efficient in relation to traffic, which may justify a more costly structure at one place than at another point where its location would involve disproportionately increased operating expenses, are the principal factors emphasized in the paper presented before the Mechanical and Engineering Section of the Franklin Institute, which is reprinted below in slightly condensed form. The permanency of the river bed and the character of the foundations are also very important considerations which should be thoroughly investigated.

LENGTH OF SPANS

Where no limitation is placed by the Government as to length of all spans, the spans should be made of economical length, provided the piers do not reduce the cross-section of the river sufficiently to cause an undesirable increase in the current velocity. This economical length may be determined by trials, and will be attained approximately when the cost of the superstructure and of the substructure are nearly equal.

In the case of the Quebec Bridge, while the navigation interests fixed the clear height of the structure above high water at 150 ft., the length of span is entirely due to the physical conditions of the crossing. The stream at this point is narrow and deep, the depth in the center of the stream being about 190 ft. The current velocity at ebb tide is very high—about 9 mines an hour. Very heavy ice runs at times and tends to gorge. The bed rock, as shown by the borings, while accessible near the shore lines, dips rapidly toward the center of the stream. All these conditions made it imperative to build a span of great length. The information as to bed rock which we now have would indicate that the original project could have been designed with a somewhat shorter span. Yet we should remember that this original project was undertaken by a private corporation, and we should perhaps recognize the value to it of such advertisement as the building of the longest span in the world would obviously afford. The next longest span is that of the Firth of Forth Bridge, and is 1700 ft. long. It is doubtful if a shorter span than 1700 ft. would have been practicable at the location adopted for the Quebec Bridge. I consider it perfectly legitimate to build a more expensive structure than economy of the work itself would call for, if the more expensive structure will afford sufficient advertisement and publicity to compensate for the additional expenditure. Cases also often arise where a purely economical and utilitarian structure would be entirely out of harmony with the surroundings.

A project to build a large bridge at Quebec, presumably in the same location as the present one, was seriously considered in 1884 and 1885. Messrs. James Brownlee, A. Luders Light, and T. Claxton Fidler designed a structure with a clear span of 1442 ft. The description of that project mentions rock foundations. The more complete information we now have, and which was obtained by a costly series of borings, shows that at the present location rock could not have been attained in both piers with any known method of foundation if the piers had been spaced only 1442 ft. apart, even if the great depth of water could have been overcome.

It may be remembered that after the disaster of Aug. 29, 1907, the Dominion Government took up the reconstruction of this bridge. A board of three engineers, including myself, was appointed to design and construct the bridge. After some study of the situation, the board decided that the new bridge should be made wider between trusses and designed to carry heavier loads than those originally contemplated; that, further, none of the old steel work could be used to advantage. It also decided to keep the same location. The final outcome is a double-track span 1800 ft., with a width of 88 ft. between centers of trusses, designed to carry on each track a live load consisting of two E 60 engines placed in any position in a train weighing 5000 lb. per foot so as to produce greatest strains. The old piers were not large enough for the new design and could not, therefore, be used. At first the board contemplated building an entirely new pier 57 ft. south of the present north pier, and enlarging the foundation of the south pier by sinking additional caissons adjacent to the old caisson. The necessary span length would then have been 1758 ft., and it was on that length of span that tenders were asked. It developed later, from the experience of sinking the north cassion, that the method proposed for enlarging the south foundation would not be safe, even if it were practicable, and so an entirely new foundation and pier were decided on for the south shore. The new north pier could not be placed farther out in the river because of the sloping bed rock and great depth of water. The south pier could not be placed on the north, or river, side of the old south pier because of the old wreckage, so it was placed 64 ft. 8 in. south of the old pier, or as close as possible to it. Both new piers being placed 64 ft. 8 in. south of the old piers, measured between centers, the new span remains 1800 ft. long.

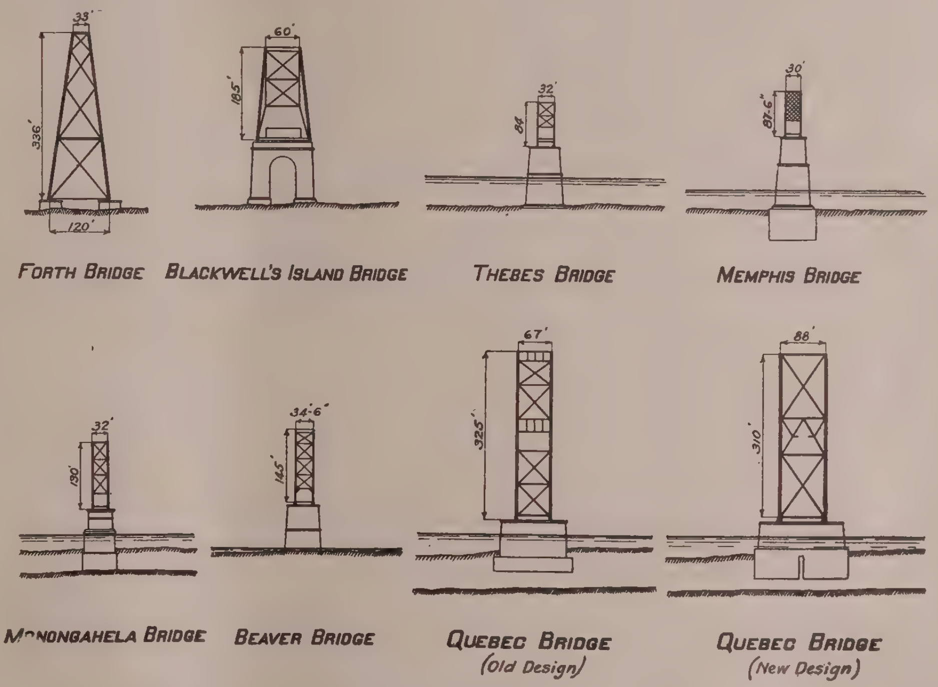
SUBSTRUCTURE

The piers are all of granite backed with concrete. There is an increasing tendency now to build everything of concrete. Certainly, concrete is a most convenient material and quite economical. When it comes, however, to providing supports for a very important and expensive structure, cut-stone masonry should be used in preference to concrete, except for backing. There are many varieties of excellent building stone on this continent. I have used granite, some varieties of oolitic limestone, also sandstone, which all show excellent lasting qualities in works constructed many years ago, while concrete presents some uncertainties and requires expert care to give good results. Concrete may in ages prove to be as lasting as stone masonry, but as yet we do not know. We know that well-constructed stone masonry will last for centuries. A notable example of this is the great Aqueduct of Gard, built by the Romans in the first century B. C.

Types of Superstructure.—Having fixed the span lengths of a bridge, the next thing to determine is the type of superstructure to be used. The various types usually applied to long spans may be classified as follows: (1) Arches (steel)—(a) three-hinged; (b) two-hinged. (2) Simple spans. (3) Cantilever structures—(a) with suspended span; (b) without suspended span. (4) Suspension bridges.

I have purposely omitted masonry and concrete arches as structures not coming clearly within the scope of this paper. Steel arches have a rational application only where Nature has provided natural abutments, as at Niagara Falls, for instance, or where natural surroundings lend themselves to ornamental construction.

Crooked River Arch.—I have recently completed an arch bridge for the Oregon Trunk Railway in the Crooked River Canyon. It is a two-hinged spandrel braced arch. Three-hinged arches are now seldom used for a railway bridge because they are less rigid than two-hinged arches. They still have a good application in roof trusses which are not subject to heavy and rapidly moving loads. In



Main Pier Vertical Posts or Towers for Longest Cantilever Spans several large arch spans the central connection offered considerable difficulties. The reason is that the top chords at the center of a purely two-hinged arch are calculated to have stresses in them, due to dead load and temperature, when the span is riveted up and ready to receive the rolling load. These stresses had to be introduced by powerful jacks, or other means, before the final point could be riveted up. To avoid this difficulty, I have proceeded as follows in the Crooked River arch. It was assumed that the dead load and temperature stresses at 60 deg. Fahr. in the top chord at center are zero. This being the case, the arch acts as a three-hinged structure under those conditions—namely, with all dead load in place and at 60 deg. Fahr. The span was, therefore, erected as a three-hinged arch, all dead load, or the equivalent, including the decking, was placed thereon, and at a time when the temperature was very nearly 60 deg. the center panel of the top chord was inserted and riveted up. The calculations were simple. The dead-load stresses were calculated as in a three-hinged arch, the temperature and live-load stresses as in a two-hinged arch, and the various results combined.

Types of Span

A two-hinged spandrel braced arch is probably the best type to use for railway traffic, where the natural abutments permit of sufficient rise, which will often be the case where the arch type of bridge is a logical solution. This particular type of arch is more rigid and less subject to vibration than the other types of arch, and presents the advantage of easier erection, which can then be simply performed by treating each half as a cantilever arm held back by suitable temporary anchorages until the center connection is made. An arch bridge is a somewhat special structure and is rarely used for very long spans, except, as I remarked before, where Nature has provided abutments.

Simple Spans.—Twenty-five or thirty years ago the system of truss most favored for long simple spans was a double intersection Pratt truss with parallel chords. Nowadays the type most used is a single intersection Pratt truss with subdivided panels and curved top chord. The curved top chord is an element of economy. The single system has also a slight advantage of more definite stresses. It has its disadvantages, such as, for instance, the lack of uniformity in deflection, about which I will speak more in detail in connection with cantilever system. There is no doubt that a bridge composed of simple truss spans is a better bridge than a cantilever system or a suspension design, chiefly because of its rigidity. This rigidity results from the fact that a load placed on one span has no lifting action on the adjacent spans, as in a cantilever system, or on other portions of the same span, as in a suspension bridge. But long simple spans must be erected on falsework or floated into position. The first method is often inadmissible on account of the necessity of keeping the channel open for navigation or excessive depth of water combined with navigation requirements, or other local conditions. The floating of a span into position is not only costly but hazardous. It has been successfully performed, but is not always feasible or safe. Then, too, generally speaking, a cantilever bridge is more economical for long spans. These considerations often lead to the adoption of a cantilever design in preference to simple spans. A cantilever span can always be erected without falsework, although the adjacent or anchor spans must generally be erected on falsework. Sometimes simple spans are erected without falsework.

The length of simple spans has been growing from year to year. It may be remembered that the Cincinnati Southern Bridge at Cincinnati, built in 1877, contains a simple span of 515 ft. In 1891 Mr. George S. Morison built the Cairo Bridge, connecting a span 518 ft. in length, single track. The Municipal Bridge in St. Louis has three simple spans of 668 ft. in length, 110 ft. high at the center, double track and roadways. The Metropolis Bridge over the Ohio River, if the present design is carried out, will have a simple span 720 ft. long, double track. This increase is due largely to the use of higher grade materials, such as nickel or chrome-nickel steel, and to the improvement in shop and field methods.

And here we may say a few words about wind forces. In small spans the action of wind rarely affects the main members of the span, and the wind bracing used is calculated more to make the structure rigid against lateral motion under rapidly moving loads than to take care of actual wind stresses. As the length of span increases, this element of wind becomes more and more important, until in very long spans it may become as important as the moving load. In a simple span the heaviest members, as well as the greatest height of truss, occur near the center of the span. In other words, the resistance to wind per lineal foot of truss in a simple span is greatest at the center of the span, and, owing to the overturning moment due to wind, grows in importance with the height. In a cantilever span the greatest height of truss and the heaviest members are near the piers, hence the greatest resistance to wind per lineal foot of truss is near the piers. This remark is sufficient to explain why wind stresses are easier to provide for in a cantilever structure than they are in a simple span of the same length.

Proportions of Simple Spans with Curved Top Chords

	Per cent ratio height to length	Per cent ratio width to length
Municipal Bridge, St. Louis, Mo...	16.47	5.24
Cincinnati, Ohio, over Ohio River..	15.41	5.60
Pennsylvania Railroad, over Delaware, Phila.	15.76	5.82
Ohio Connecting Railway, Pittsburgh	14.53	5.57
McKinley Bridge, St. Louis, Mo...	15.07	5.73
Merchants Bridge, St. Louis, Mo...	14.47	5.80
Mobridge, Mo.	15.18	5.48
New Brunswick Bridge, N. D......	16.25	5.50
Quebec suspended span...........	17.19	....
Thebes suspended span...........	15.03	....
Average	15.56	....

Longest Simple Span.—No hard-and-fast rule can be laid down as to the length at which a simple span becomes uneconomical as compared with a cantilever span. Generally speaking, considering the present knowledge of materials, a simple span of 700 ft. may be taken as the practical economical limit, beyond which it is not advisable to go without a thorough investigation and comparison with a cantilever system. Where conditions require unusual methods of erection this limit may be much lower. For instance, in the Thebes Bridge it was necessary to erect the 671-ft. channel span without falsework. A simple span would have required the addition of a considerable amount of metal, both in the span itself and in the adjacent spans, to permit of its being erected as a cantilever; this excess of metal would have been useless after the completion of the bridge, and its cost would have made the bridge more expensive than the adopted cantilever design.

Dimensions of Simple Spans.—In designing long simple spans the following general principles should be observed. The width, center to center, of trusses should not be less than one-twentieth of the span, preferably one-eighteenth. In double-track spans the width required for clearance generally governs, except in very long spans. The height at center of span, for a Pratt system of truss with subdivided panels, should be from one-seventh to one-fifth of the length. The table on this page shows the proportions of the width and height to the length of span in some of the curved top-chord bridges built recently.

The height at the ends should be only sufficient for an effective portal. A rigid mathematical research by Mr. Joseph Mayer, principal assistant engineer of the Quebec Bridge, leads to a theoretical proportion between the height at center and length of span equal to 0.18, or somewhere between one-fifth and one-sixth, for heaviest loads and double-track spans.

Panel Length.—The best panel length is not so easy to determine. Since the economical inclination of diagonals is very nearly 45 deg., it would result that in a Pratt truss with subdivided panels their length should approach one-half of the height of truss. This would mean that in a truss with a curved top chord the panels in the center should be longer than those near the end. This has been done at least in one instance, namely, in the Municipal Bridge in St. Louis. The advantages of such an arrangement are a slight economy in the weight of steel and an improved appearance, since the diagonals have nearly the same inclination throughout. The equal panels, however, present, in my opinion, two decided advantages, which may more than offset the advantages of the former system, namely, that, all panels being equal, there is a greater duplication of parts, the floorbeams are all alike, except at ends, the stringers are all alike, the length of bottom chord eyebars is the same throughout; and, further, that the falsework may be built in uniform panels, and the traveler, which is usually designed with a view to have the uprights in proper relation to the panel points, so that the connections may easily be made, preserve its relation to the various panels points as it is moved from panel to panel. For these reasons uniform panels should be preferred in the majority of cases of simple spans.

Maximum Span Lengths

I have already stated that the greatest practical length of a simple span is about 700 ft. With the use of certain known alloys of steel of greater strength than medium carbon steel this limit may become as much as 750 ft. Beyond this limit the weight of simple spans becomes so great, in comparison with cantilever spans, that the latter must be considered. A mistaken idea sometimes prevails that the weight of steel in a span increases in proportion to the square of the length. This is, in a measure, true for short spans, say 100 to 300 ft. This ratio of increase, however, is not a constant, but increases with the span. A simple span above 1200 ft. in length increases in weight approximately as the cube of the length, and this exponent increases more and more rapidly until at about 2000 ft. the weight of carbon steel required to carry the weight of such a span and of a moderate live load becomes infinite. For a span built of nickel steel the weight becomes infinite when the length reaches 2700 ft. Simple spans much below those limits, even if possible, would still be very uneconomical until we get down to spans 700 ft. or under.

Cantilever Spans.—This leads us to cantilever spans. I mentioned two types of such spans—one without a suspended span and the other with a suspended span. A remarkable example of a cantilever bridge without a suspended span, which may be called a semi-continuous structure, is the Blackwell's Island, also called the Queensboro Bridge, in New York. There seems to be no advantage in omitting the suspended span; on the contrary, the structure differs from a true continuous bridge over several supports only by the introduction of a hinge at the center of the main span which transmits shears but not moments. The vibrations and deflections of each segment are, therefore, transmitted through those hinges to all the other segments. Furthermore, since the stresses in such a structure depend on deflections, there is more or less uncertainty in the calculations. I do not wish to be understood as objecting to any type of structure seriously, because of the uncertainty of calculations. In any logical construction the calculations can always be made with sufficient accuracy for the safety of the work. It is only when everything else is equal that determinate stresses should be preferred.

Let us consider the usual type of cantilever bridges, the one in which two cantilever arms support a suspended span. We may assume that in bridges requiring the construction of a cantilever span the length of the main span is usually determined by local conditions. The general dimensions to be fixed by the designer are, therefore, the length of the suspended span, the length of the anchorage spans, when these are not determined by local conditions, the height of the trusses at various points and the relative distances and positions of trusses to one another. Let us discuss these various dimensions in connection with the new Quebec design. The Quebec Bridge, with its longest span in the world, has justly attracted much attention among engineers and has naturally elicited comment and criticism. It is acknowledged that a discussion of a scientific subject by professional men is often of greater value than an elaborate paper on this same subject by one individual. If I refer to some of the criticisms, let it be considered as a friendly discussion which may be of value to the profession.

NEW QUEBEC BRIDGE

The new Quebec Bridge has been finally designed with two anchor arms 515 ft. long, a suspended span 640 ft. long, and two cantilever arms 580 ft. long. The moving loads finally adopted for the Quebec Bridge are: On each track two Cooper's Class E-60 engines, followed or preceded, or followed and preceded, by a train load of 5000 lb. per foot per track. In addition to the actual dead load of the structure, a load of 500 lb. per lineal foot on suspended span and 800 lb. on balance of bridge was allowed for snow.

Wind Loads.—The wind loads were taken as follows: A wind load normal to the bridge of 30 lb. per square foot of the exposed surface of two trusses and one and a half times the elevation of the floor (fixed load), and also 30 lb. per square foot on travelers and falsework, etc., during erection.

A wind load on the exposed surface of the train of 300 lb. per lineal foot applied 9 ft. above base of rail (moving load).

A wind load parallel with the bridge of 30 lb. per square foot acting on one-half the area assumed for normal wind pressure.

In the Forth Bridge the enormous wind load of 56 lb. per square foot was assumed. This load was imposed on the designers by the Board of Trade soon after the Tay Bridge disaster. The Tay Bridge was not designed to withstand even a 30-lb. pressure. This assumption of a 56-lb. wind in the Forth Bridge results in a very large addition of metal in the bottom chords through which the wind stresses are transmitted to the piers. The material in those members is distributed as follows:

	Gross tons
Dead load	2282
Live load	1022
Wind load	2920
Total	6224

The metal here provided for the wind is nearly three times that provided for the live load, and is about 47 per cent of the total required.

In the new Quebec Bridge design the wind pressure is equivalent to about 35 per cent of the uniform live load near the piers and to about 20 per cent of the live load near the ends of the cantilever arms.

A pressure of 30 lb., according to German experiments with electric cars, would correspond to a wind of a velocity of over 100 miles per hour. Other experiments made at various times on small surfaces show that a velocity of 85 miles would correspond to a pressure of about 30 lb.

The following formula for wind pressures is generally used: $P = kv^2$, in which P = pressure per square foot, v = velocity in miles per hour, and k = a coefficient.

Eiffel's two hundred or more experiments show this coefficient to vary from 0.0026 to 0.0032, and the average is 0.0030, which he recommends. Trautwine makes $k = 0.0050$, which seems too high. But, even using the latter, a pressure of 32 lb. would correspond to a "hurricane" of a velocity of 80 miles. The German experiments agree with Eiffel's. Making $k = 0.0030$, a pressure of 30 lb. would correspond to a velocity of 100 miles per hour, which, according to Trautwine, is a violent hurricane uprooting "large trees."

With a wind of this velocity there would be no traffic on the bridge—empty freight cars or even light passenger cars would be overturned. Velocities of more than 85 miles may occur in cyclones and tornadoes over restricted areas. Such storms are very rare in Canada; but even should such an extraordinary disturbance happen, causing a wind pressure of as much as 60 lb. to be applied to the entire Quebec Bridge as now designed—the stresses in the truss members would be less than with the maximum live load and a 30-lb. wind—and although the stresses in the laterals would be increased above the specification limits, they would still remain within the elastic limit of the members.

Length of Suspended Span.—The length of the suspended span does not depend merely upon the most economical distribution of material required for carrying the live loads and the dead load of the bridge after it is completed. Where there are no other considerations beyond the actual working stresses in the finished structure, the most economical length of the suspended span for a total span of 1800 ft. would be in the neighborhood of 1000 ft. But to erect a simple span of such unprecedented length, either by floating or by cantilever method, would be impracticable. Furthermore, the cantilever method of erecting a suspended span of even a moderate length always requires additional material, both in the cantilever arms and in the suspended span, to take care of the erection stresses.

The longer the suspended span in relation to the total main span, the greater will be the required addition—so that whether it be contemplated to erect the suspended span by cantilever method or by floating into position, the length of the suspended span finds itself limited not by mere economic considerations of the finished bridge, but by either the excess of material required during erection by cantilever method, and difficulties arising therefrom, or by the difficulties attending the floating of a very long and heavy span into position. These difficulties increase very rapidly with the length of the span to be floated. In the new design the suspended span is the longest which the board considered safe to float, and it fits the entire design very well.

ERECTION BY FLOATING

The erection of this span by floating made it possible to design it with the view to greatest economy. Its various members will not be subjected to any greater stresses during erection than they would be in a simple span of the same length resting on two piers. It was, therefore, possible to design it as economically as to weight as a well-designed simple span would be. It is more important to save weight in a suspended span than in an independent simple span, because each pound in the former requires several pounds in the entire structure to carry it. The importance of economy in the suspended span of the Quebec Bridge will be appreciated when it is considered that one pound uniformly distributed over the trusses of the suspended span needs 3 lb. of metal added to the bridge to carry it, making an addition of 4 lb. in all. This accounts for curved top chords in the span in question, as well as for the use of nickel steel for the trusses thereof.

Length of Anchor Arms.—It has been pointed out that the length of the anchor arms is uneconomical—that a shorter arm would have been cheaper. It must not be forgotten that a shorter anchor arm increases the pier reactions, as well as the steel in the anchorage proper. The present anchor piers are founded on rock ledges which dip rapidly toward the river. To move them nearer to the river would have involved much more expensive foundations.

It may be remarked here that, while an addition of dead load in the main span will require several times the weight of metal to carry it, an addition of dead load in the anchor arm requires no increase of metal to carry it when there is an upward or negative reaction on the anchor pier. This is explained by the fact that any load placed between the main piers or on the main spans increases all moments and shears over all the spans, while any load placed on the anchor arm, if the reaction on the anchor pier is negative, decreases that reaction and consequently the moments in the anchor arm, but has no effect whatever on the main span. For this reason carbon steel will be used mostly in the anchor arm of the new design. The carbon-steel unit stresses adopted for the Quebec Bridge are generally five-sevenths of the nickel-steel stresses, the former requiring heavier members. This additional weight in the anchor arms is a source of economy when the relative prices of carbon and nickel steel are considered.

Height Over Piers.—An opinion has been expressed that the height over the piers of the new Quebec Bridge is not great enough for economy. Actual calculations show that for economy the height of 310 ft. in the Quebec design is too great by about 20 ft. for the "K" system of trussing adopted; further, that this height would have been at least 40 ft. too great for the original system of the official design. The height of the Forth Bridge towers, while 26 ft. higher than the Quebec Bridge, though the span is 100 ft. shorter, is no doubt economical for the form of trussing adopted for it. The economical height is not only a function of the length of the span but also of the panel length next to the pier. This height should be such as to correspond to an inclination of the diagonals not far from 45 deg. A double-intersection system with very long panels near the pier, such as adopted in the Forth Bridge, would have been economical for the Quebec Bridge, except that it requires a system of secondary members or sub-posts, or very heavy longitudinal girders, or both, to carry the load from panel to panel. Then, too, it is well to reduce in the members the stresses due to their own weight—which in long panels become quite important. The 20-ft. excess in height of the present Quebec design over what would have been the economical height is justified by the resulting reduction in the sections of the bottom chords, which are of considerable size at best.

Straight versus Curved Chords.—In long cantilever spans the bottom chords of the cantilever and anchor arms should be straight when possible. With a curved chord the joints must be made at the panel points. These joints are of great importance, as has been shown in the report of the Royal Commission on the Quebec Bridge disaster. They should be fully spliced to take care of secondary stresses due to deflections of the span during erection and under the action of live load. It is advisable, therefore, to place them outside of the point of connection with the diagonals and keep them clear of gusset plates. The same objection does not exist in top chords of simple spans, which are of moderate sizes, even in the longest spans known. The economy in simple spans resulting from such curved chords is worth while and quite important, while if any economy were to result from curving the bottom chord of the cantilever and anchor spans, such economy would certainly be of little importance in comparison with the resulting disadvantages. The vertical deflections from live loads are not as great in a straight chord design as in a curved chord design.

Another consideration in favor of the straight chords is that the most important, in fact the bulk, of the wind forces travel to the pier through the bottom chords of the cantilever and anchor arms and the wind bracing, or lateral system situated in their plane. The straight bottom chords carry these stresses direct to the piers without transmitting any appreciable components to the web system of the trusses. Not so with curved bottom chords. At each point where the chord's direction is changed a component stress is transmitted to the web. This means that while a pair of straight chords with its lateral system deflects under the action of the wind in the plane of the chords only, a pair of curved chords, by transmitting shear to the web members, causes the trusses to deflect, the windward truss downward, tending to flatten the curve, and the leeward truss upward, tending to make the curve more pronounced. The rigidity of the straight chord design against lateral deflections and oscillations is therefore greater than that of the curved chord design.

One of the reasons why curved bottom chords were used in the cantilever arms of the original Quebec Bridge design was the fact that it was the aim of that design to provide full headroom of 150 ft. on a width of 1000 ft. The bottom chords of the anchor arms were then made curved also for the sake of symmetry. The width on which the full headroom will be obtained has been reduced in the new design to about 760 ft., which certainly is more than ample to accommodate navigation. Only the highest vessels will be limited to this width of 760 ft., and that only at high water.

The top chord of the Quebec Bridge cantilever and anchor arms is straight. The Forth Bridge cantilever arms have straight top chords also. While there was good reason for making the Forth Bridge top chord straight, there was no serious reason, beyond a slight increase in vertical rigidity, for making it straight at Quebec. The two trusses on the Forth Bridge are in planes inclined toward each other at the top. The two top chords are parallel. Had they been made curved they could not have been parallel, since they must necessarily be situated in the inclined planes of the trusses. The appearance of tension chords having a greater distance apart at the center of the arm than at either end would have been very bad. But there is no such reason at Quebec. The trusses are in vertical planes and the top chords could have been curved without serious inconvenience, but also without any advantage. The board considered that, aside from the additional vertical stiffness, a straight chord will present an appearance of strength which a curved chord would not do.

(To be concluded)

How Not to Build a Retaining Wall

By L. E. Moore, Assistant Professor of Civil Engineering, Massachusetts Institute of Technology

The writer has been spending some time at Falmouth Heights, Mass., and has observed with interest a retaining wall which is in the course of construction at that point. The wall is at the foot of a bluff about 25 ft. high and is intended to arrest the crumbling and washing down of the material in the bank. The wall is about 8 ft. high and extends 4 or 5 ft. above the level of the beach, with the inclined face against the bank. The resistance of the beach to the sliding of the wall should not be counted upon, as it is cutting away quite rapidly under the action of the waves.

The cross-section of the wall is shown in Fig. 1. If the earth pressure be assumed to act at the center of the back of the wall, which is fair in view of the heavy surcharge, and if the usual principle that the resultant pressure on the portion of the wall must not be outside the middle third be followed, we have the following interesting conclusions based on a length of 1 ft. of wall: Taking the weight of concrete as 150 lb. per cubic foot, we have a total weight of wall for 1 ft. in length of 3200 lb. This weight acts through a point 13⅔ in. in from the face of the wall, or 2⅓ in. outside the middle-third point of the base. The thrust P of the earth back of the wall acts with a moment arm of 40.6 in. about the outer middle-third point. That is to say, this wall violates every principle of good design and it is impossible to make the resultant pressure come within the middle third of the base.

Had the wall been turned around with the vertical face against the bank, the condition of affairs would have been as shown in Fig. 2. Taking moments about the outer middle third point of the base we find the P can be equal to 1222 lb. before the pressure falls outside the middle third.

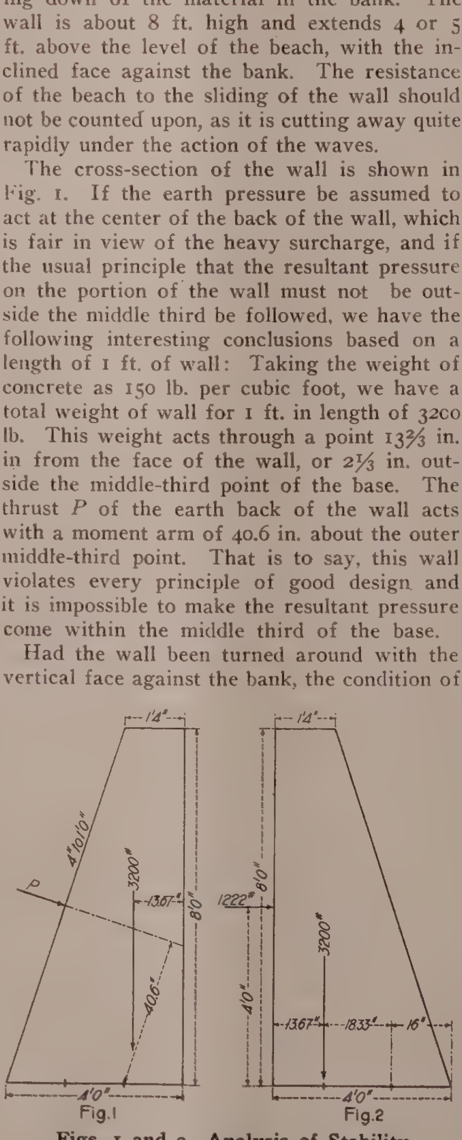


Fig.1 Fig.2

Figs. 1 and 2—Analysis of Stability

The history of this wall is that a portion was built two or three years ago of a different cross-section and fell the succeeding winter. It was rebuilt and buttressed as shown in the photograph, Fig. 3. Of what use is the part of the buttress above a line drawn from the intersection of the wall and the upper surface of the buttress down to the intersection of the end of the buttress and the ground? I am informed that the town of Falmouth has appropriated $6,500 to complete this wall, which will be of very doubtful strength when it is completed. Had the wall been built the other side out the expense would have been no greater and a stable structure would have resulted. A very small portion of the cost of the wall spent in securing sound engineering advice would have been an excellent investment.

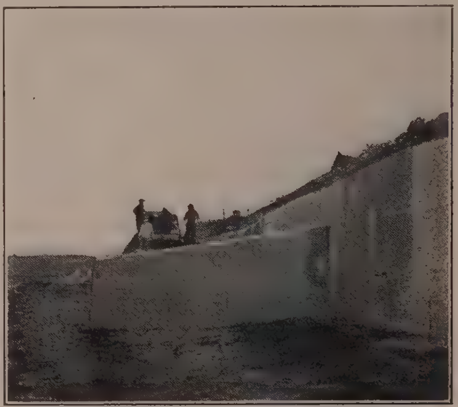


Fig. 3—Buttress of Doubtful Economy

Highest Railways in the World

In the spring of 1912 there was opened in South America a railway between the harbor town of Arica in Chili and La Paz, the capital of Bolivia. The gage is 3.28 ft. (1 m) and the Andes Mountains are passed through a tunnel located 13,986 ft. over the ocean. After passing the tunnel the railroad follows the Maure River, crosses the Desuguadero River and terminates at La Paz, which is located at an elevation of 12,113 ft. The total length of the road is 300 miles.

This railroad, however, is not the highest in the world. In the fall of 1912 the meter-gage Antofagasta Railway was opened, connecting the Mulati River with Potosi. This road has its highest point at El. 16,006.

Of standard-gage railways the Peruvian Central Railroad is the highest. It connects the harbor city Callao with the capital Lima and the city of Oroya. Its tunnel through the Andes Mountains is located at a height of 15,750 ft.

In Europe the highest railroad is located on the Jungfrau Mountain in Switzerland. This mountain is 13,665 ft. high and the highest point reached by the road is 11,140 ft.

In the United States the 9-mile railroad up Pikes Peak in Colorado reaches the top of the mountain, which is 14,216 ft. high.

HYDRAULIC BUFFERS have been installed in the new railroad station at Pretoria, according to the "South African Railway Magazine." They are designed to stop a train weighing 300 tons running 6 miles per hour.

Calculation of Docks and Bulkhead Walls

Discussion of Forces and Loads and Designs of Structures Necessary to Resist Them

Docks and bulkhead walls are generally designed, so far as provision against the forces due to the earth back of them is concerned, largely on empirical grounds, the construction following the lines of structures which experience has shown are satisfactory. In Germany more attention has been given to theoretical considerations than in this country, but even German books do not generally treat the subject other than superficially. For this reason, though the address was delivered several years ago, translation has been made of a lecture by Prof. Wolmar Fellenius, the well-known Swedish harbor expert, before the civil engineering faculty of the Royal Technical University of Stockholm, Sweden. So far as Professor Fellenius and the Engineering Record are aware the address has not previously been translated into English.

EARTH PRESSURES

The methods herein described are based on the theory of earth pressures, in which it is assumed that (1) the earth is a homogeneous mass without cohesion, and (2) that in moving the earth slides on the plane of maximum pressure, or as it also is called the plane of rupture. Although these assumptions, especially the first, do not conform to what really takes place, they are used because the whole earth-pressure theory is based on them.

Active earth pressure is the pressure which the earth exerts on a fixed or elastic support, while passive earth pressure is the resistance which the earth opposes to a body pressed against it. Generally, engineers are inclined to underestimate the influence of such passive pressures, but for harbor structures, as will be seen in the following discussion, it is of the utmost importance to consider them in the calculations.

For various angles of the plane of rupture with the horizontal the earth pressure has different values and it is, therefore, essential that this angle is assumed so as to give the maximum active pressure; for the passive pressure, however, the angle ought to be assumed so as to produce the minimum pressure. Furthermore, to be on the safe side, the maximum active pressure should be increased, say by 25 per cent, and the passive lessened by 25 per cent, if the angle of repose of the earth has not been assumed lower than that to be expected.

Earth and water pressures are considered as working in unison. To this end two assumptions are in general use:

1. The total pressure of earth and water is taken as the pressure of the earth only, where the weight of the earth per cubic foot is equal to the weight of a cubic foot of earth saturated with water.

2. The total pressure of earth and water is taken as the full water pressure plus the earth pressure, where the weight of the earth is the weight minus the weight of water it displaces.

The latter condition probably prevails if the earth consists of previous material—stone, gravel, sand, etc.—but the first assumption is justified if the earth is impervious—soft clay, etc. As it is impossible to insure against water penetrating between the support and the earth, and as the consideration of the separate pressures gives a higher value for the total pressure that theory ought to be used under all circumstances.

The simplest kind of a dock or harbor structure is an anchored sheet piling, as shown in Fig. 1. After the depth of the basin, the dock level and the position of the anchor have been assumed, the depth to which the sheet piling has to be driven must be calculated. For this purpose the following graphical construction is recommended: First, the earth and water pressures against the sheet piling are determined down to a greater depth than that expected for the sheet piling. The diagrams for the normal components against the sheeting are drawn and eventually the active pressure is increased 25 per cent and the passive pressure decreased by the same percentage. If, now, the passive pressure is drawn on the same side of the wall as the active the load diagram shown by the hatched area in Fig. 2 will be obtained. This diagram is divided into subareas representing forces, and force and line polygons are constructed. It is to be noted that the forces in the load diagram act in two different directions—against the inside of the piling from 1 to 5 and against the outside from 6 to 9. From the intersection between the first line in the line polygon and a normal to the sheet piling at the point where the anchor is placed a tangent to the line polygon is then drawn. The point tangency indicates the depth to which the piling must be carried to insure stability.

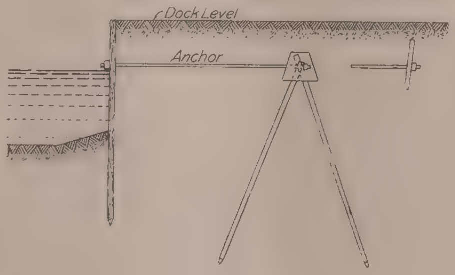


Fig. 1

If, now, from the pole in the force polygon a line is drawn parallel to this tangent the normal component *S* acting on the anchor is determined. That this method is correct is shown as follows: If the sheet piling is carried down to the point of tangency the tangent is the last line in a line polygon drawn on the forces acting on the piling, and consequently the resultant of all of these forces must go through the intersection. Furthermore, as this resultant balances all of the acting forces it is evident that the piling is carried down to a sufficient depth.

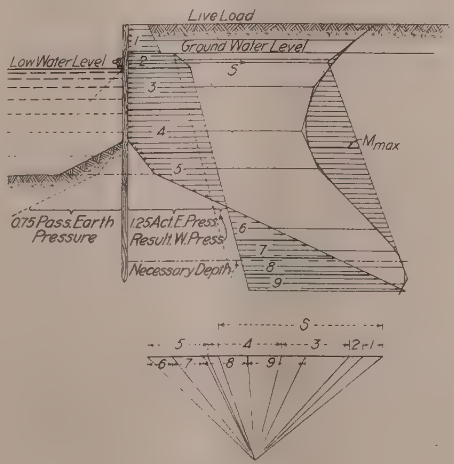


Fig. 2

The moment area for the piling is also given on this diagram, from which the maximum moment is easily determined by drawing another tangent to the line curve parallel to the first tangent.

Should the anchor be secured to pile bents, the pull in it is divided into components along the two pile directions, and from this the push and pull on the piles are calculated. Should the anchor, however, be connected to an anchorage of sheet piling or a pressure-distributing plate, the resistance is calculated as the difference between the passive earth pressure on the outside of the plate, without any live load on the dock, and the active pressure on the inside of the plate, with live load on the dock. The point of application of the anchor to the pressure distributing plate should conform with the point of application of the resultant of the pressures.

The anchorage plate ought to be placed far enough from the sheet piling so that the plane of rupture for the active earth pressure, acting on the sheet piling, will intersect at or above the dock level the plane of rupture for the passive pressure acting on the anchorage plate.

DOCK STRUCTURES ON PILES

In a dock structure like that shown in Fig. 3 sheet piling is generally placed either (1) at the front or (2) at the rear of the masonry. In both cases the piles supporting the structure decrease the load on the sheet piling. In case (1) this is accomplished by the piles reducing the active earth pressure back of the sheet piling and in case (2) by increasing the passive earth pressure on the outside of it. The best way of considering the influence of the piles on the ground is to use for the material through which the piles are driven a larger angle of repose than that assumed for material in general. After this slope has been determined the sheet piling can easily be calculated by the methods outlined above. This being determined, the piles under the masonry must be examined to determine whether they prevent the moving of the earth between them. If they are able to hold back the earth its angle of repose will be increased. One method of figuring this, when sheet piling is used in the rear, is to assume the resistance of the piles as the resistance of imaginary sheet piling at the same place as the other sheeting subjected to the same passive earth pressure so far as the existing sheeting reaches, plus the active earth pressure below this point. These two pressures will be resisted on the other side of the piling by passive earth pressure calculated for the angle of repose (not the increased) of the earth outside of the sheet piling.

It is evident that the piles must reach at least to the required depth for such imaginary sheet piling and be able to take the same maximum moment. If the piles are of about the same diameter, this moment can be equally divided among them.

The masonry part, therefore, in addition to the dead and live loads, and the pressure from the earth back of it, is subjected to horizontal forces from the sheet piling and from the piles. The first of the latter forces is known as soon as the sheet piling is calculated and the other from the analysis of the imaginary sheet piling.

When all the forces acting on the masonry part are known they are combined and their resultant, *R*, is determined as to amount and direction. If all of the piles are parallel with the direction of *R* and the center of gravity of the piles conforms with the point of application of the resultant R, the load on each pile is easily determined. Should these conditions not prevail, more or less complicated calculations must be adopted.

Calculating the Pile Foundation

A method often used for calculating the pile foundation is to divide the resultant into its horizontal and vertical components and distribute the vertical as though the structure were supported on an elastic foundation. The diagram for this resulting pressure is then divided so that each pile takes an amount of vertical load corresponding to the spacing. The horizontal component is either neglected or when there are batter piles combined with the horizontal components due to the vertical loads. The rest of the horizontal forces are then generally taken care of by anchors or special tension piles.

These methods, however, often lead to absolutely impossible results. The method described below has, therefore, been devised, based on the following assumptions:

1. That the masonry body is so inelastic that its change in shape can be neglected as compared with the change in shape of the piles—that is, that a horizontal plane at the lower surface of the masonry after deformation of the piling is still a plane.

2. That the load per pile is in proportion to its longitudinal deformation.

These assumptions correspond to those in the bending and deflection theories and their use here is, therefore, justified.

In calculating the loads on the piles, it is further assumed that due to the load R, the masonry takes a new position determined by a horizontal motion, x, a vertical, y, and a rotation δ. This rotation has to be taken around a fixed point on the under side of the structure; for instance, the lower front corner. The angle α between the center line for each pile and the x-axis; the resistance coefficient, k—i.e., the resistance in the pile against a deformation (elongation or compression) equal to the length unit; and the distance a between the assumed point of rotation and the intersection of the center line of the pile and the under side of the masonry are determined, and by neglecting the change in direction of the pile due to the deformation the motion in its longitudinal direction is

$$\Delta = x \cos\alpha + (y - a\,\delta) \sin\alpha$$

(see Fig. 4)

and its corresponding load is

$$P = k\,[x \cos\alpha + (y - a\,\delta) \sin\alpha] \qquad (1)$$

As the position of the resultant R is known—its inclination being β and its distance from the point of rotation b—the following equations can be written:

$$R \sin\beta = \Sigma\, k\,[x \cos\alpha + (y - a\,\delta) \sin\alpha] \sin\alpha$$
$$R \cos\beta = \Sigma\, k\,[x \cos\alpha + (y - a\,\delta) \sin\alpha] \cos\alpha$$
$$R\,b \sin\beta = \Sigma k\,[x \cos\alpha + (y - a\,\delta) \sin\alpha]\,a \sin\alpha \qquad (2)$$

From these equations x, y and δ are determined and the load on the piles P calculated by equation (1).

It is not necessary to know the exact values of the coefficients k; it is sufficient to know the relations between them. For instance, assume $k_1 = c_1 s$; $k_2 = c_2 s$, etc., where s is an unknown constant, and c_1, c_2, etc., are known coefficients.

If these expressions are introduced in the equations (2) x, y and δ are solved as functions of s in the form of $x = A/s$, $y = B/s$ and $\delta = C/s$, where A, B and C are, from the equations, known quantities.

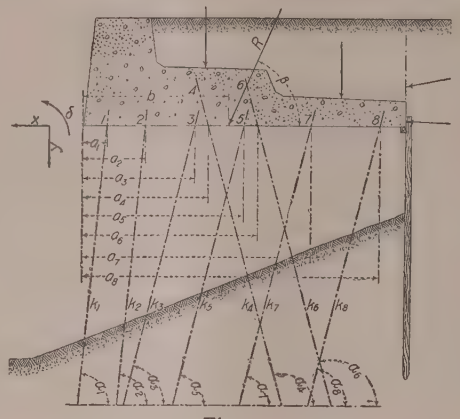


Fig. 3

By introducing these values in the equation

$$P = c\,s\,[x \cos\alpha + (y - a\,\delta) \sin\alpha]$$

the unknown s disappears and the P receives a fixed value.

Should c have a different value for push and pull—which, generally, is the case—these have to be assumed and afterwards the result checked against these assumptions and the necessary adjustments made.

For friction piling the values of c can be assumed to be in proportion to the friction surface of the respective piles, but for tension piles only three-quarters of this surface should be considered in the calculation.

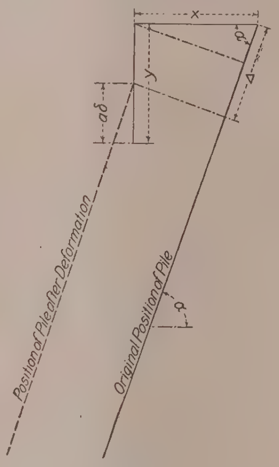


Fig. 4

It is of interest to examine the equations (2) when $\beta = 90$ deg. and $\alpha_1 = \alpha_2 = \ldots = \alpha_n = 90$ deg.

The only unknown quantities are then y and δ, and the equations then read

$$\left.\begin{aligned} R &= \Sigma\, k\,(y - a\,\delta) \\ R\,b &= \Sigma\, k(y - a\,\delta)\,a \end{aligned}\right\} \qquad (3)$$

and if k is equal for all piles

$$\left.\begin{aligned} R/k &= \Sigma\, y - \delta\,\Sigma\, a \\ R\,b/k &= y\,\Sigma\, a - \delta\,\Sigma\, a^2 \end{aligned}\right\} \qquad (4)$$

If a and b are now figured from the center of gravity for the entire pile foundation,

$$\Sigma\, a = 0 \text{ and } \Sigma\, a^2 = I$$

With the number of piles equal to n the equations then become

$$R/k = n\,y \text{ and } R\,b/k = -\delta\,I$$

or

$$y = \frac{R}{k\,n} \text{ and } \delta = -\frac{R\,b}{kI}$$

and from this expression of the pile load

$$P = k\,(y - a\delta) = \frac{R}{n} + \frac{R\,b}{I/a} \qquad (5)$$

which formula corresponds to the one in use for the pressure distribution on surfaces subjected to eccentric loading.

Improving the Foundation

Should the ground under the structure be poor—that is, consist of soft clay or mud of a considerable depth—it is often found impossible to design a structure in which piles and sheet piling are kept within economic limits. In such cases it is advisable to improve the foundation by dredging and depositing gravel into which the piles are afterwards driven. In this case the structure can be calculated as entirely founded on gravel, providing the original ground is able to support the fill. Should the ground be too soft to do this the fill will sink more or less unevenly until it reaches firmer layers. It is impossible in such a case to design the gravel fill theoretically and the engineer is entirely thrown upon his experience in building such structures. But, if the original ground is able to support the fill, it can be theoretically determined in the following way:

Fig. 5 shows the general arrangement of such a fill and φ_c = angle of repose of clay above ground-water surface; φ_c' = angle of repose of clay below ground-water surface; γ_c = weight of clay per cubic foot above ground-water surface; γ_c' = weight of clay per cubic foot below ground-water surface; φ_g = angle of repose of gravel above ground-water surface; φ_g' = angle of repose of gravel below ground-water surface; γ_g = weight of gravel above ground-water surface; γ_g' = weight of gravel below ground-water surface.

As the dredging and filling are done under the water and the fill deposited shortly after the dredging the slopes ED and GH can have a considerably larger angle with the horizontal than φ_c'.

It is advisable to consider the lowest water surface outside of the structure in the calculations because it gives the maximum value of the active earth pressures. The ground-water surface on the inside of the structure can also be assumed as being higher than the water surface on the outside, in which case the difference between the water pressures has to be considered.

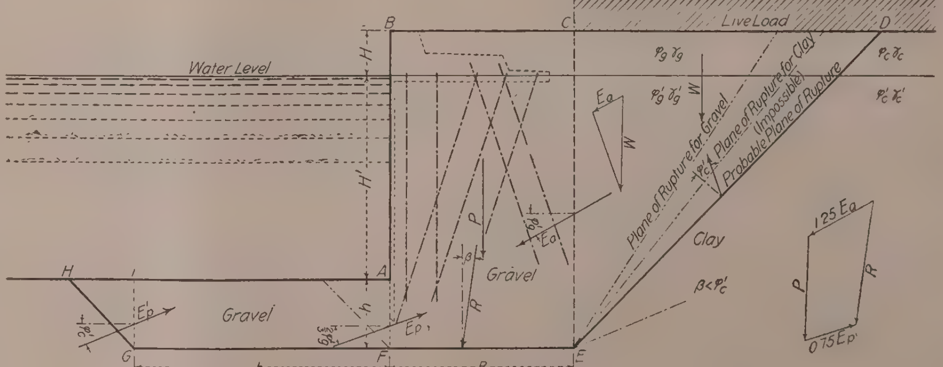


Fig. 5

The first thing to be determined is the depth *h* of the fill outside of the structure. This depth ought to be at least so large that at the lower edge of the front of the structure (point *F*) the intensity of the active pressure of the gravel on the inside plus the difference in pressure between the water surfaces on either side is less than, for instance, three-quarters of the passive pressure of the gravel on the outside.

The bottom width *B* of the fill inside of the point *F* is then determined by treating the mass *FABCE* as a body subjected to active and passive earth pressures. The maximum active earth pressure is applied at *CE*. In order to determine this pressure the plane of rupture for the maximum pressure must be known. This angle for the gravel falls generally inside of the fill—that is, it passes through the gravel, and the pressure due to it is the gravel pressure and, therefore, cannot be the maximum. In most cases the angle of the plane of rupture for the maximum pressure for the clay falls inside of the fill and is, therefore, an impossibility; generally the slope *ED* must be considered as giving the maximum pressure. With the help of graphical methods the active earth pressure E_a can then be calculated, under the assumption that it forms an angle φ_g' with the horizontal.

The passive gravel pressure E_p on the outside of *AF* is then determined, under the assumption that because of the sheet piling in or near *AF* the angle with the horizontal is only $2/3\ \varphi_g'$.

After the weight *P* of the mass *FABCE* is known it is combined with the pressures E_a and E_p to form the resultant *R*, in which, as a factor of safety, the active pressure is to be increased 25 per cent and the passive lessened 25 per cent. The resultant must then fall within the middle third of the base *B*, and its angle β with the vertical must be less than the angle φ_c'. The following approximate equation for the width *B* may serve to obtain the first trial value. In this equation the horizontal components, only, of the pressures $(E_a)_h$ and $(E_p)_h$ are used:

$$B\,[H\gamma_g + (H' + h)\,\gamma_g']\ \tan\varphi_c' \geqq 1.25\,(E_a)_h - 0.75\,(E_p)_h$$

Finally, the bottom width *b* of the fill outside of the structure has to be determined. Should this width be too small it may be that the pressure E_p does not reach its full value, in which case the fill nearest to *AF* will slide upward, on the plane of rupture for E_p and move the fill outside in an outward, horizontal direction. The reason for this is that the minimum passive earth pressure on *GI*, E_p', is very much smaller than E_p.

The following method of calculating this fill so that it will stay in place is recommended: After E_p' has been determined by the same method as is used to calculate E_a, with the change that now the minimum pressure is sought, the following equations can be used:

$$b\,h\,\gamma_g'\tan\varphi_c' \geqq (E_p)_h - (E_p')_h$$

As a factor of safety in this equation, especially if the depth *h* is small, the term E_p' may be neglected.

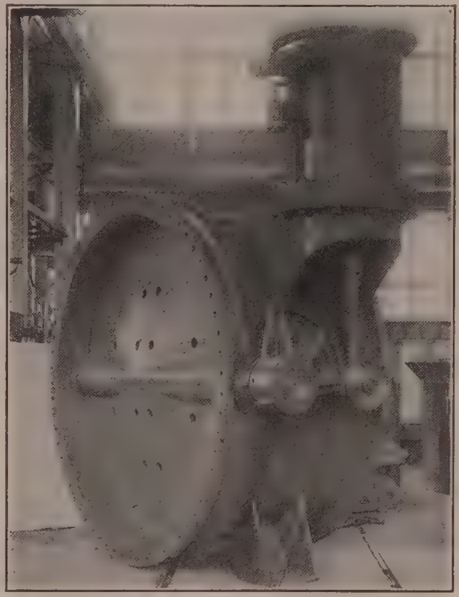
This equation is very simple and is founded on the principle that the friction between the fill and the clay must be able to balance the difference between $(E_p)_h$ and $(E_p')_h$.

From the above calculations the dimensions of the gravel fill are arrived at, but it should be observed that in many cases by increasing the depth *h* the other dimensions of the fill are decreased, thereby lessening the cost of the whole structure.

Butterfly Gates for the San Francisquito Power Station No. 1

Butterfly gates, 84 in. in diameter, are located on the steel penstocks of the San Francisquito Power Station No. 1 about 70 ft. from the center of the surge chamber (see Engineering Record, Aug. 23, page 218, and Sept. 13, page 299). These gates will enable repairs to be made on the penstocks and will serve as an emergency should a break occur in the pipe line. The closing time will be about two minutes.

They are of the Pelton design, modified by the addition of a packing ring. This ring is of bronze, in two sections, and properly adjusted by fifty-four alternating cap and jack screws spaced around the circumference of the valve casing. After each jackscrew is adjusted in its countersunk hole the hole is closed by a pipe plug screwed up tight on the head of the screw. When closed the disk stands in a vertical position, necessitating these refinements so as to insure watertightness.



Butterfly Valve Assembled

Placed on a cantilever bracket on one side of the casing is the operating device, consisting of a 24-in. cylinder having a 2 ft. 4¼-in. stroke. The 3-in.-diameter piston rod is connected below the cylinder to a rack working against a sector, keyed onto the shaft of the gate, to revolve the disk. The pressure in this cylinder is applied by oil under a static head of 130 ft. from a tank located on the side of the hill. Connected to the cylinder is a common governor pilot valve controlled by two solenoids. In these solenoids is a soft Norway iron core, which has a lever connection to the piston rod of the pilot valve. By letting a 110-volt current pass through one of the solenoids the core will be attracted, thus operating the pilot valve and guiding the pressure oil either above or below the piston in the cylinder. These solenoids are controlled from the power station; they are each 8¾ in. long and 4¾ in. in diameter; the core is 1¼ in. The device is 20½ in. in length.

Opening the valve will be acomplished by hand manipulation of an oil line valve, located so that the operator can see the sector and thus fill the penstock by just "cracking" the gate. The penstocks may also be filled at the power house by cross-connections between the exciter supply lines.

Carriers' General Suggestions for Railroad Valuation

In connection with the general subject of railroad valuation a sheet of suggestions in diagrammatic form as to inventory and information to be obtained under the valuation act has been issued by Mr. Thomas W. Hulme, general secretary of the Presidents' Conference Committee.

The first item suggested is improvements, with an analysis of the methods by which original cost with additions, cost of reproduction and cost of reproduction less depreciation are obtained, and the reasons for their differences, if existent. Supplemental to this it is recommended that other values and elements of values be analyzed as to methods of valuation employed, and reasons for any differences between such values and the foregoing costs.

Land, right-of-way and terminals used for common-carrier purposes are proposed for consideration next, original cost being compared with present value, and in the case of land acquired by condemnation the original cost is to be compared with the present cost. The third classification pertains to property held for purposes other than for those of a common carrier, and with such property it is proposed that original cost and present value be obtained, with an analysis of the methods of valuation employed.

Fourthly, financial matters are recommended for consideration, subdivided into (1) history and organization of present and previous corporations operating any part of the property; (2) increases or decreases of stocks, bonds or other securities in reorganization; (3) moneys received by any of the corporations by reason of any issue of stocks, bonds or other securities; (4) moneys for syndicating, banking or other financial arrangements under which such stocks, bonds or other securities were issued, and the expense thereof; (5) gross and net earnings of each present and previous corporation; (6) expenditures of all moneys in such detail as the Interstate Commerce Commission may require, and the purpose thereof; and (7) such other elements as the commission may require to ascertain the original cost to date.

In the fifth and last group are placed for consideration the amount and value of any aid, gift, grant of right-of-way or donation made to the reporting carrier or to any predecessor by any government, individual or others; grants of land and the amount derived from the sale of portions thereof; value of the unsold portion at the time of acquirement; value of the unsold portion at the present time; and the amount and value of any concession and allowance made to the United States or to any state, county or municipal government in consideration of any aid, gift, grant or donation.

Horse Down a 30-Inch Manhole

A horse hitched to a heavy truck at Rochester, N. Y., recently caught his shoe on a manhole lid, pulling the cover up as he lifted his forefoot. His hind legs sank through the opening, only 30 in. in diameter, and after many kicks and struggles the animal freed himself from his harness and gradually slipped into the opening until head and forefeet had completely disappeared below the surface of the street. A derrick was summoned, and after much difficulty the horse was hauled out by ropes looped around his neck and front feet. With the exception of a few scratches the horse apparently suffered little from his close contact with 2500-volt cables in the manhole.

Substructure for East River Bridge Division, New York Connecting Railway

Building High Concrete Piers and Retaining Walls and Deep Caisson Foundations for Three and One-Half Miles of Four-Track Viaducts and Bridges

The East River Bridge division of the four-track $25,000,000 New York Connecting Railway is about 3½ miles long, and consists entirely of steel bridges, plate-girder viaducts with high concrete piers, and embankments between high retaining walls, as described in The Engineering Record of July 19, 1913. Most of the methods of operation involved in the execution of the work are of a standard character, and none of the concrete work, except that for the west tower of the Hell Gate arch bridge, is of exceptional nature. It is of interest on account of the selection and arrangement of plant, transportation and handling of materials and its unusual magnitude.

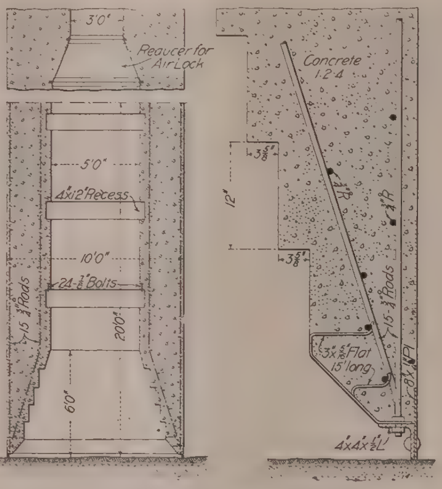
The Eastern Viaduct

The Eastern viaduct, 3480 ft. long and from 35 to 60 ft. high, consists chiefly of plate-girder spans and retained fill, with concrete arches and lattice-girder spans over intersecting streets. This construction requires about 70,000 yd. of concrete, 180,000 yd. of fill and 10,000 tons of structural steel. The open-pit foundations are carried a short distance to hard stratum and the concrete work is built in several simultaneous 200-ft. sections, with separate concrete plants commanded by tower and chute installations at their central points.

Sand, gravel and broken stone are unloaded from scows by clamshell buckets, which deliver to five 5-ton Garford and two 6-ton Saurer motor trucks of 4½-yd. capacity. These transport them about 2 miles, at the rate of sixteen loads each per ten-hour day, to a point near the center of the section, where the trucks dump into depressed wooden pockets, from which the material is delivered by a derrick with clam-shell bucket dumping either on large storage piles or into elevated storage bins, from which it is chuted to the automobiles for delivery to the concrete plants as required. In most cases the automobiles deliver direct from the scows to the concrete plants.

Concrete Mixing and Placing

There are two principal concrete plants, in each of which a Hains mixer is provided with a Dempcy-Degener bucket elevator, which raises the aggregate to the top of the storage bin over the mixer about 30 ft. above the surface of the ground. Stone is first chuted into the measuring hopper, cement is added and finally the sand for a 17.5 cu. ft. batch, after which the required amount of water is very quickly admitted through a 3½-in. outlet and the contents of the hopper are discharged through the mixing hopper to the bucket below. Great pains have been taken to secure the best possible results with this machine, and different sequences of charging and different quantities of materials have been tried, none of which gave as good results as those above mentioned. The mixer is not as satisfactory when an amount either greater or less than the rated capacity of the machine is charged, and accuracy of regulation and uniformity of addition of the water are considered important.



Details of Working Chamber and Cutting Edge of Caisson

The concrete bucket is hoisted to the top of a guyed wooden tower, where it automatically dumps into the highest of a series of steel chutes, which extend to a maximum length of 200 ft. in both directions from the tower, and discharge directly into the forms. The first section of these chutes leads to an auxiliary center tower on the axis of the railroad, from which the successive sections are swung in both directions to the opposite walls of the structure. The sheet-steel chutes are 10 in. wide and at frequent intervals are suspended by manila-rope tackles from overhead steel cables. They have a minimum inclination of 1:3, except for lengths of more than 150 ft., when it is 1:2½, and have delivered a maximum of 260 yd. per ten-hour day per machine.

The outside-wall forms are made of steel plates stiffened with exterior flange angles, and the inside forms are simple wooden panels. They are connected by removable ¾-in. horizontal tie rods engaging waling pieces which are bored to receive them, besides which there are diagonal wires in transverse vertical planes, which are adjusted to line up and plumb the forms and prevent them from warping. There are two main towers about 150 ft. high, each with its concrete mixer, and three auxiliary towers about 40 or 50 ft. high, without special mixers, which are supplied from a movable mixer shifted as needed. The main towers command a length of structure equivalent to about one block, and after it is completed they are taken down and re-erected in advance, and so on.

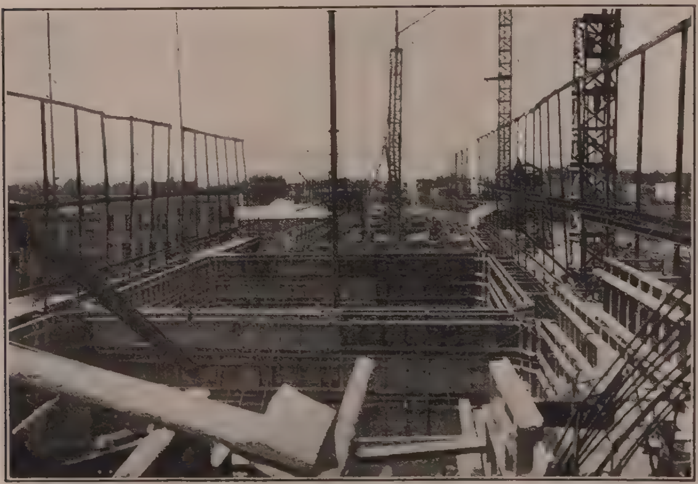
Plate-Girder Viaducts and Piers

The Randalls Island, Wards Island and Long Island Viaducts, all of plate-girder construction, have a combined length of 7478 ft. and have concrete piers from 60 to 130 ft. in height, with footings built in open pits. Separate concrete plants are established for each viaduct, and the work on all of them is proceeding simultaneously, and involves an aggregate of about 165,000 yd. of concrete.

Sand and gravel are delivered in barges, unloaded by a clamshell bucket and transported in 4-yd. dumpcars drawn, for the Long Island Viaduct, by dinkey locomotives over an elevated trestle and discharged through it into hopper-bottom bins. From the bins the aggregates are chuted into bincars of 7-yd. capacity, which take them in bulk, together with cement in bags, to the mixing plants, where they are chuted through gates in the cars to the measuring hoppers, and thence charged to the machines, which deliver directly to buckets hoisted to the tops of the concrete towers at different heights up to 230 ft. From these towers the concrete is chuted into the forms.

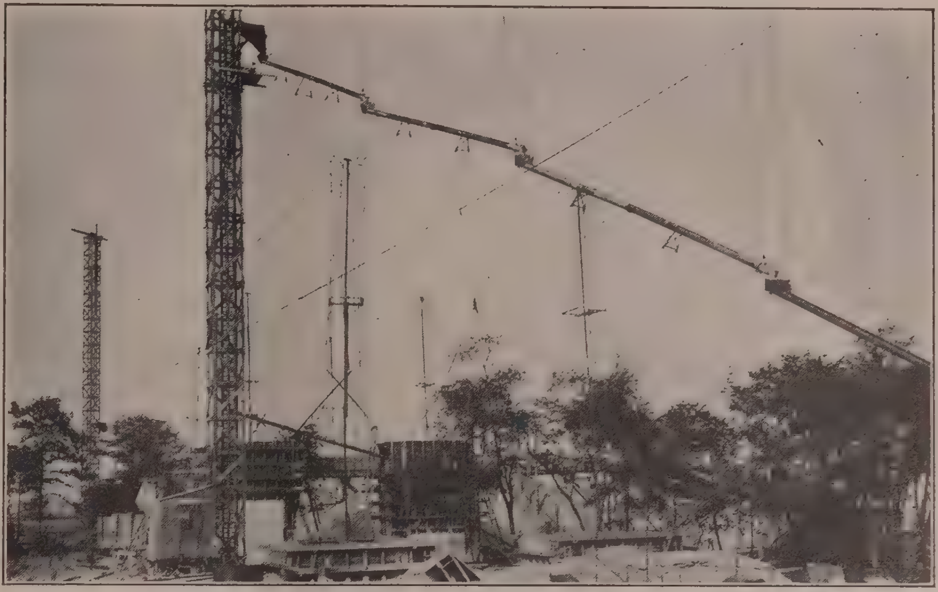
Hoisting and Chuting Towers

The towers, about 5½ ft. square, are made with four 6 x 6-in. corner posts, with horizontal and diagonal bracing on all faces. They are bolted together in the yard in sections 14 ft. high and are erected as units by a ginpole



Thin Retaining Walls Connected by Transverse Diaphragms

Arch Reinforcement and Forms for Concreting, Eastern Viaduct



Tall Concrete Chuting Tower for Randalls Island Viaduct Piers

braced to a horizontal bottom sill and having a horizontal cantilever arm at the top from which tackles are suspended. The ginpole, about 30 ft. high, is carried up as the tower progresses, and after the tower is completed is left securely fixed at the top, ready for the removal of the tower. Sheaves set in the double horizontal arm provide for the concrete-bucket hoist line. The towers on this section are supplied with Lakewood machines and a third movable Lakewood mixer is operated in advance of the towers to mix the concrete for the foundation in the bottom of advance piers beyond the reach of the tower chutes.

The main installation is on Wards Island, where the west tower for the Great Hell Gate span is located. Sand, gravel and broken stone are unloaded by clamshell derricks and delivered to a Robins 24-in. inclined belt conveyor about 250 ft. long, which rises to an elevation of about 50 ft. above the surface of the ground and dicharges into three elevated hopper-bottom bins, two of them for stone and one for sand. Bags of cement are also delivered by the elevator and are chuted down a reversed incline to the mixing platform. The bins, of 2000-yd. capacity, deliver the aggregate through chutes to 18-yard bin cars, which supply Lakewood concrete mixers, as already described for the Long Island plant. The mixing machines deliver to buckets hoisted to the tops of towers like those on Long Island and the concrete is delivered in chutes from the tops of the towers.

MAIN SPAN FOUNDATIONS

The 100 x 140-ft. towers extending 250 ft. above the ground surface for the 1000-ft. four-track Hell Gate arch span will contain 110,000 yd. or more of concrete mixed by the plants already described, that are installed on Long Island and on Wards Island. On Wards Island the depth of the rock, which is believed to vary from about 70 to 140 ft. below mean low water, makes the construction of the pier foundation very difficult and costly. The foundations will consist of three ranges of cylindrical concrete caisson piers, each having five units 18 ft. in diameter and 20 ft. apart on centers longitudinally, which will receive vertical loads, and three 26 x 125 ft. rectangular concrete piers concentric with the alignment of the bridge trusses and proportioned to receive both vertical and horizontal components of the load. Each rectangular pier will consist of three separate caisson units connected and bonded together by vertical keys.

This arrangement will enable the different portions of the foundations to be sunk only as far as is required to reach satisfactory bearing in different portions of the tower area. The caissons will be made of reinforced concrete having steel cutting edges and sunk at first by excavation under water, with orange-peel dredge buckets operated through the interior open wells of the caissons.

At the tower site, close to the edge of the river, a large amount of ground water is encountered near the surface, and it is expected that a deep stratum of large boulders will be found within 20 or 30 ft. of the surface, beyond which it is anticipated that there will be fine sand down to solid rock, which test borings indicate to be from 70 to 140 ft. below mean low water, thus making a maximum excavation of 150 ft. or more not improbable. It is hoped that this can be accomplished entirely by the open dredging method; but provision is made for changing to pneumatic-caisson work if necessary. The circular caissons are now being constructed at the site in forms made with wooden vertical staves hooped with angle-iron rings. The pier site is near the storage plant referred to. A mixing plant, with distributing tower and chutes, has been installed immediately adjacent to the storage bins to be used in furnishing concrete for the caissons.

The viaduct and pier construction on Wards Island include about 60,000 yd. of concrete, besides about 72,000 yd. required for the substructure of the Little Hell Gate and Bronx Kill Bridges and the Randalls Island viaduct, for which work considerable use will be made of locomotive cranes, one of the Brownhoists being used on Wards Island.

BRONX VIADUCT

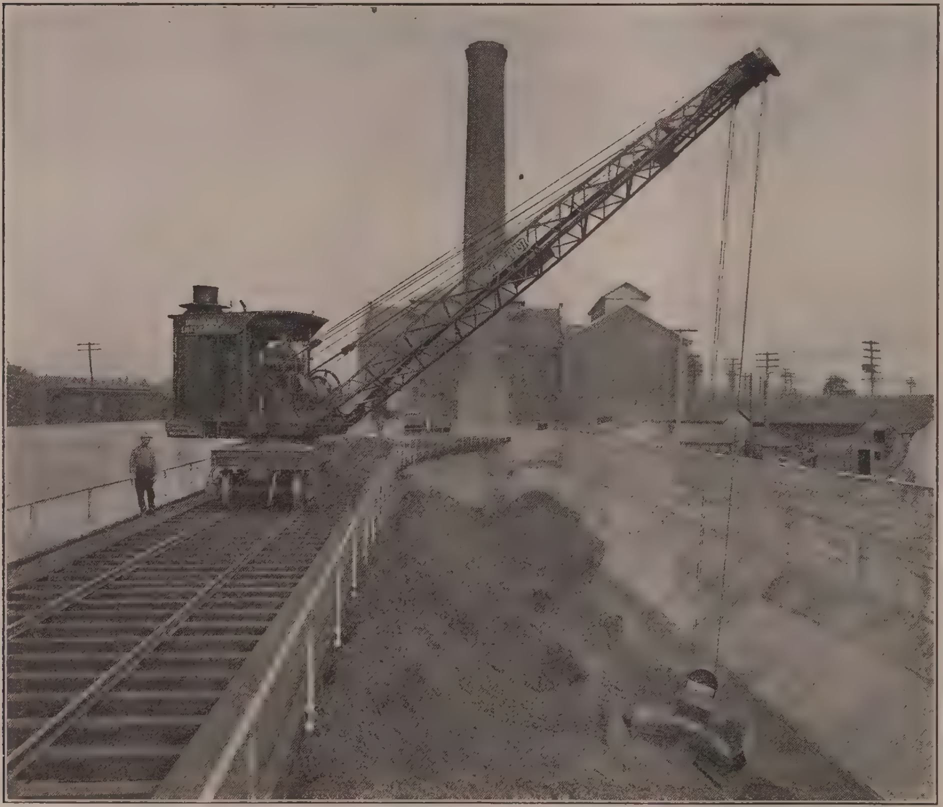
The 4200-ft. Bronx viaduct, 20 to 60 ft. high, is composed of plate-girder spans on concrete piers, with caissons, piles and open pit foundations, and about 800 lin. ft. of fill between high buttressed concrete retaining walls, together with a small amount of unretained embankment with sloping sides. The principal quantities involved include 45,000 cu. yd. of concrete and 30,000 cu. yd. of fill and embankment.

Concrete materials delivered in scows were unloaded by a clamshell bucket into 4-yd. dump cars drawn by four 3-ft. gage locomotives to the storage piles, where they were dumped from the top of inclined planes. The clamshell buckets operated by guyed derricks delivered the sand and gravel from the two storage piles to 10-yd. hopper bottom bins over the mixers in the concrete plant. Buckets of concrete were delivered on flat cars on the narrow-gage service tracks to three Brownhoist and one Ohio locomotive crane with 50-ft. booms that traveled alongside the work on the standard-gage siding of the De Lavergne Machine Company, which was located parallel and adjacent to the right-of-

Wooden Forms and Arch Centers for Randalls Island Viaduct

Steel Forms for Concreting Randalls Island Viaduct Piers way of the connecting railroad and enabled it to be commanded by the crane.

The retaining wall had a maximum height of 38 ft., with a base 14½ ft. wide and 7 ft. thick, on a wooden-pile foundation. The wall, 2 ft. 6 in. thick at the base and 1 ft. 3 in. thick under the coping, is reinforced by a vertical sheet of triangular mesh-steel fabric and by 1-in. horizontal bars 12 in. apart on centers. It is stiffened by reinforced-concrete buttresses 18 in. thick in vertical planes and 8 ft. apart on centers. The footing concrete was placed in wooden forms and the walls above were built in courses of convenient height in forms made with steel plates reinforced by outside flange angles for the exterior face of the wall and by wooden panels for the inner face and buttresses.



Locomotive Crane and Grab Bucket for Handling Coal at Storage Bin

Caisson Foundations

The piers for the plate-girder spans of the viaduct are supported on ninety-one cylindrical caisson foundations, 10, 12, 13, 15, 16, 17 and 18 ft. in diameter, sunk to an average depth of 40 ft. and a maximum depth of 55 ft. They have angle-iron cutting edges and vertical and horizontal reinforcement of rods ¾ in. in diameter, and were built in steel forms on the surface of the ground. They were sunk by dredging through the center well with a ½-yd. Smith orange-peel bucket. They were sunk from 8 to 12 ft. through fill to blue clay by hand excavation, the groundwater below 3 ft. from the surface being removed without difficulty by ordinary Cameron pumps. The fill was underlaid with a 25-ft. stratum of blue clay, which sealed the caissons and excluded most of the ground water, enabling the work to be carried on by hand if desired; but it was found generally more desirable to excavate there with the orange-peel buckets. Below the clay the caissons were sunk through 8 to 10 ft. of quicksand into hardpan or rock stratum. As soon as they entered the quicksand the shafts were closed with air locks and the remainder of the sinking was carried to completion under pneumatic pressure.

As the work progressed the height above the cutting edge at which the air locks for the successive caissons were required could be closely estimated in advance, and at that level the fifteen vertical reinforcement rods terminated, with two ends engaging the bottom flange of a conical riveted steel reducer 3 ft. in diameter at the top, to which the Moran air lock was connected. Three reducers and two air locks were supplied and were shifted from caisson to caisson as the work advanced. Usually two caissons were under air pressure at once and were sunk without difficulty by their own weight and by cast-iron blocks piled on them around the air-shaft. Whenever the caissons showed a tendency to get out of plumb they were easily corrected by placing one, two or three inclined wooden struts against the down-hill side of the caisson and securing them rigidly to it; as the sinking continued, the struts, being fixed at the lower ends, forced the caissons back into the vertical as they revolved through arcs of circles.

Concrete was mixed in two plants, one of which was equipped with a Garlin cube mixer and one with a Smith machine. The forms were assembled in 5-ft. courses, one or more of which up to a maximum height of 25 ft. were concreted at one operation. The work has been carried on with a maximum force of about two hundred and fifty men.

The contractor for the Bronx viaduct is also building the piers for the Bronx Kill Bridge with ten pneumatic-caisson foundations varying from 24 x 24 ft. to 31 x 36 ft. The caissons are built on two-part wooden pontoons, which are afterward removed and the floating caissons finished, placed in position, and sunk in the usual way. Compressed air for this work, as well as for the land caissons, is furnished by three low-pressure compressors, which, together with two high-pressure compressors, are installed in a central steam-power plant, from which the air is distributed in both directions along the line of work in a 6-in. low-pressure and a 3-in. high-pressure pipe. The work is being executed by an average force of about two hundred and fifty men and is handled without special difficulty other than that imposed by the very restricted space available for the plant.

Mr. Gustav Lindenthal is the chief engineer; Mr. O. H. Ammon, assistant chief engineer, and Mr. H. W. Hudson, engineer of foundations and masonry in direct charge of design and construction. The contracts for the work here described, commencing with the Bronx viaduct, are being executed by Arthur McMullen, the Patrick Ryan Construction Corporation and the T. A. Gillespie Company.

Plant for Subaqueous Coal Storage

In the coal-storage plant recently erected by the Indianapolis Light & Heat Company, of Indianapolis, provision is made for submerging the coal under the water to prevent spontaneous combustion. The plant has a total storage capacity of 30,000 tons, providing against coal shortage due to strikes at the mines or on the railroads and to car shortage on the railroads.

The plant consists of a large concrete pit, as shown in the illustration, 300 ft. long by 100 ft. wide and 28 ft. deep. It is 18 ft. below ground level, with a wall rising 10 ft. above the ground. Below the ground the wall was built at a slope of 45 deg. to reduce the cost of construction. This makes the pit 72 ft. wide by 272 ft. long on the bottom. The concrete is from 12 to 18 in. thick and is reinforced throughout with twisted ½-in. square rods to withstand any pressure from water that may rise in the gravel around the pit.

Along the longitudinal axis of the pit there is a row of concrete piers placed on 15-ft. centers. These piers support a trestle on which there runs a standard-gage track. A 15-ton Brownhoist locomotive crane travels back and forth on this trestle, handling the coal on both sides with a Brownhoist 2-cu. yd. coal grab-bucket, suspended on a 40-ft. boom. The coal cars are run out on the trestle with the crane and dumped; when the coal is required in the power house it is loaded into cars by the crane and carried to the boilers.

Forty-pound rails are embedded in the bottom of the pit, on 18-in. centers, with ½ in. of the head standing above the surface of the concrete. These rails prevent the bucket from striking the concrete when working on the bottom of the pit. The pit is filled with water up to the ground level, covering 13,000 tons of coal. The water is pumped from the powerhouse supply well into the pit by a 4-in. centrifugal pump. Through the center of the pit and beneath the trestle there is a trough 12 ft. wide and 1 ft. deep, which serves as a drainage channel. It is fitted with a drain pipe which carries the water to either of two manholes one at each end of the pit. The outlet openings are protected by gratings to prevent any coal from entering the drain pipe.

The pit was designed by Mr. Thomas A. Wynne, vice-president of the company, and was built under his supervision.

Six Large Timbers for Dredge Spuds were recently shipped from the Portland Lumber Company's mill at Portland, Ore., for use in Government dredges on the Panama Canal. To of these are 30 in. square and 66 ft. long, and four are 24 in. square and 70 ft. long. All have been carefully selected and are said to be unusually fine specimens.

Fresh Sludge and Decomposed Sludge

Comparative Analyses and Discussion of Sludge-Utilization Possibilities

By Hermann Bach and Leslie C. Frank, of the Emscher Drainage Board, Germany

It is intended in this article to present a comparison, partly based upon the experiments described, of fresh sludge and decomposed sludge as obtained from Imhoff tanks. The first object of the experiments was to obtain data regarding the result of the process taking place in Imhoff tanks, for although much has been written in the past regarding Imhoff tanks and the results of their operation very little is known about what actually takes place in them beyond the fact that there is practically no development of H_2S manifested and that the other gases evolved are non-odorous. The second object of these experiments was to obtain data upon which to base a comparison of fresh sludge and decomposed sludge from the standpoint of utilization. It is claimed occasionally not only that sludge utilization is feasible, but that to neglect it is a deliberate waste of valuable material.

Many modern engineers, however, doubt the possibility of utilizing sludge from an economic standpoint, except in some special cases, and are of the opinion that the engineer engaged in the construction or operation of a sewage disposal plant should not be misled by theoretical expositions of the question of utilization, but should lay emphasis only upon the simple disposal of sludge in the cheapest and most unobjectionable manner. It is probable that in many cases the work of sewerage engineers would be facilitated if they were not forced by public opinion or higher officials to complicate their problem with the added question of utilization. When, in some special cases, valuable elements, such as fats from wool wastes, are contained in the sludge this fact will be evident immediately to the trained sewerage engineer. The composition of the sewage will also indicate to him whether it will be worth while to try to recover these valuable elements. Such special cases as these will be found in some industrial districts, but purely domestic sewages are not worth investigation in this regard.

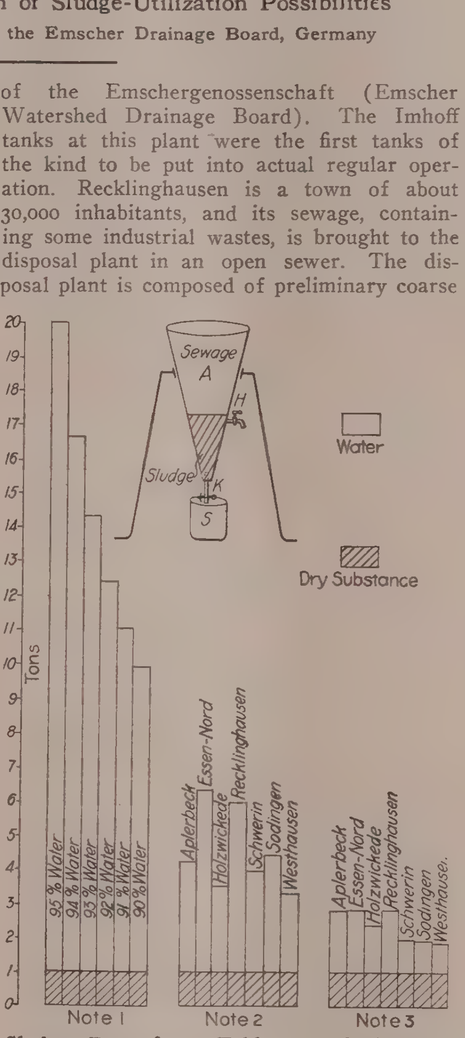
Whether one believes, however, that sludge is or is not utilizable, it will, nevertheless, be interesting to know whether sludge such as that produced by Imhoff tanks is better or worse from the standpoint of utilization than fresh sludge.

EXPERIMENTS

In 1910-11 experiments were conducted at the disposal plants at Holzwickede, Schwerin and Recklinghausen to determine the difference in chemical composition of fresh and decomposed sludge. Average daily samples of fresh sludge were taken on a different day each week, so as to determine as nearly as possible the average composition of the fresh sludge for the given period. Then, from the operation books of each plant, it was possible to determine very closely when the sludge of this period was about to be withdrawn from the tank. At this time average samples of the decomposed sludge were taken. It can scarcely be doubted from this method of conducting the experiments that the average samples of fresh and decomposed sludge were really comparable.

A short description of each plant is given below with the account of each experiment.

Recklinghausen-Ost Experiment. — The plant at Recklinghausen-Ost is the oldest plant of the Emschergenossenschaft (Emscher Watershed Drainage Board). The Imhoff tanks at this plant were the first tanks of the kind to be put into actual regular operation. Recklinghausen is a town of about 30,000 inhabitants, and its sewage, containing some industrial wastes, is brought to the disposal plant in an open sewer. The disposal plant is composed of preliminary coarse screens, a detritus pit and six Imhoff tanks. There are two sedimentation chambers, one to each three sludge chambers. The total capacity of the two sedimentation chambers is 9749 cu. ft., or 72,920 U. S. gal. The average daily dry-weather flow is 2,280,000 gal. The detention period is forty-five minutes. The chamber holds six months' sludge.



Sludge Data from Table 6 and Sampling Apparatus

Note 1.—Fresh sludge, showing per cent of water.
Note 2.—Liquid decomposed sludge placed upon drying beds.
Note 3.—Drained spadable sludge taken from drying beds.

TABLE 1—COMPOSITION OF THE SEWAGES OF THE VARIOUS PLANTS AT WHICH THE EXPERIMENTS WERE CONDUCTED (Parts per Million)

	Recklinghausen		Schwerin		Holzwickede		Essen-Nord	
	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.	Inf.	Eff.
Suspended matters—								
Total	440	126	504	123	400	45	751	164
Mineral	181	35	207	51	206	16	363	84
Volatile	259	91	297	72	194	29	388	80
Dissolved matters—								
Total	937	935	1276	1111	934	772	...	1375
Mineral	705	719	801	800	655	625	...	1268
Volatile	232	216	465	311	279	147	...	107
Chlorine	205	199	220	217	132	119	...	482
Nitrogen—								
Total	37	37	71	60	35	18	41	41
Free NH_3....	25	26	44	42	18	8	28	29
Organic	12	11	27	18	16	3	13	12
Nitrates + nitrites	...	...	...	...	1	7	...	...
H_2S in solution	10	7	10	8	6	0	11	5
Cubic centimeters per liter of sludge capable of settling in two hours	5.28	0.14	5.1	Tr.	4.0	0.1	14.8	Tr.

Inf., influent; Eff., effluent; Tr., trace.

The composition of the sewages of this and the other plants is shown in Table 1, the figures being average values of many analyses. It will be noted from this table that the reduction in total suspended matters in Recklinghausen is 71.4 per cent, while the reduction in suspended matters capable of settling in two hours is 97.2 per cent.

Twenty-one examinations were made of average daily samples of fresh sludge arriving at this plant. The samples were taken from Oct. 17, 1910, to March 29, 1911. Each hour the same quantity of the influent was taken as it left the detritus tanks. At the end of the day the supernatant liquid was poured off, the sludge placed in an airtight container, sent to the laboratory and examined immediately. According to the operation books of the plant, the decomposed sludge corresponding to this fresh sludge was that discharged from the tank between April 20 and May 11, 1911. Within this period sludge was discharged at five different times, and each time average samples were taken and examined. Table 2 gives the results.

TABLE 2—ANALYSIS OF RECKLINGHAUSEN-OST SLUDGE DRY MATTER IN PER CENT

	Fresh sludge, average of 21 analyses	Decomposed sludge, average of 5 analyses*	Increase or decrease
Mineral matters ..	30.2	45.9	+ 34.2
Organic matters ..	69.8	54.1	— 22.5
Fats..............	11.1	8.1	— 27.0
Nitrogen..........	2.4	2.3	— 4.6
†Fixed carbon....	18.0	18.4	+ 2.7
†Volatile matters.	51.8	35.7	— 31.1

*After 4 to 6 months' storage in the sludge chambers.
†Test similar to those in coal examinations.

Schwerin Experiment.—This plant takes domestic sewage from 3000 people and consists of a coarse preliminary screen and two Imhoff tanks. Originally there were also two sprinkling filters and one final settling basin, but their operation has been discontinued. The detention period is from two to three hours. The reduction of total suspended solids as shown in Table 1 is 75.6 per cent., while the reduction in solids capable of settling in two hours is practically 100 per cent.

Five examinations were made of daily-average samples of fresh sludge. The samples were taken between April 3 and June 17, 1911. According to the books of the plant, all of the sludge corresponding with this period was discharged May 1, 1912, at which time an average sample was taken and examined. The smaller plants in the Emscher district are so designed and operated that sludge can be discharged at longer intervals than in the case of the large plants. Table 3 gives the results of these examinations.

TABLE 3—ANALYSIS OF SCHWERIN SLUDGE DRY MATTER IN PER CENT

	Fresh sludge, average of 5 analyses	Decomposed sludge, 1 analysis*	Increase or decrease
Mineral matters ..	46.2	54.9	+ 15.9
Organic matters ..	53.8	45.1	— 16.2
Fats	9.5	4.2	— 55.7
Nitrogen	1.6	1.5	— 6.2
Fixed carbon ..	25.9	24.5	— 5.4
Volatile matters.	27.9	20.6	— 26.1

*After 11 to 13 months' storage in sludge chambers.

Holzwickede Experiment.—This plant receives about 80,000 gal. of domestic sewage daily from 3500 inhabitants. It consists of two Imhoff tanks and one sprinkling filter. One of the tanks is used for preliminary clarification and the other for clarification of the sprinkling filter effluent. This is the only plant of the Emschergenossenschaft making use of biological purification. The preliminary tank has a detention period of from one to two hours. The effluent from the tank is pumped to the sprinkling filter, which is of the revolving arm type, and the filter effluent then passes through the final Imhoff tanks into the stream. The sprinkling filter is composed of slag, 30 to 70 mm. in size. The decrease in total solids (see Table 1) is 88.8 per cent and the decrease in solids capable of settling in two hours is 97.5 per cent.

Ten examinations were made of daily-average samples of fresh sludge. The samples were taken between Jan. 1 and July 30, 1911. The corresponding decomposed sludge was discharged upon Oct. 31, 1911, and at this time an average sample was taken and examined. The results follow:

Table 4—Analysis of Holzwickede Sludge Dry Matter in Per Cent

	Fresh sludge, average of 10 analyses	Decomposed sludge, 1 analysis*	Increase or decrease
Mineral matters ..	43.7	69.2	+ 36.8
Organic matters ..	56.3	30.8	— 45.2
Fats.......... ..	8.9	5.5	— 37.2
Nitrogen..........	1.7	1.0	— 41.1
Fixed carbon......	18.8	11.7	— 37.3
Volatile matters..	37.5	19.1	— 49.1

*After 7 to 10 months' storage in the sludge chambers.

Essen-Nord Experiment.—The same experiments were also carried out at the largest plant of the Emschergenossenschaft, namely, Essen-Nord. This plant receives sewage from 180,000 people, as well as a large part of the Krupp and other factory wastes. One of the factories discharges iron salts, which bring about a voluminous precipitation in the sewage. The dry-weather flow equals 185 gal. per second, or 16,000,000 gal. per day. In times of storm the plant takes care of three times the dry-weather flow, the excess up to six times being received by a storm-water basin, while the excess over six times discharges directly into the stream.

The plant is composed of one coarse screen, one three units detritus tank, nineteen Imhoff tanks, one storm water basin, sludge pumps and sludge drying beds. Six of the Imhoff tanks are of the radial-flow type, and the remaining twelve of the horizontal-flow type. The total sedimentation chamber capacity is 88,980 cu. ft. The dry-weather detention period is about one hour. The sludge-room capacity is 229,500 cu. ft. The storm-water basin capacity is 67,090 cu. ft. The sludge beds cover an area of 73,150 sq. ft. The decrease in total suspended solids (see Table 1) is 78.2 per cent. and the decrease in solids capable of settling in two hours is practically 100 per cent.

The conditions of operation at this plant made it rather difficult to be sure that the samples of fresh and decomposed sludge corresponded with each other. Errors from this source were minimized as far as possible by making a large number of examinations of the decomposed sludge, and it is probable that the data in the following table are comparable:

Table 5—Analysis of Essen-Nord Sludge Dry Matter in Per Cent

	Fresh sludge, average of 15 analyses	Decomposed sludge, average of 71 analyses*	Increase or decrease
Mineral matters ..	50.9	59.2	+ 14.0
Organic matters ..	49.1	40.8	— 16.9
Fats..............	9.5	4.3	— 54.7
Nitrogen..........	2.0	1.5	— 25.0
Fixed carbon......	14.2	14.9	+ 4.7
Volatile matters..	34.9	25.9	— 25.8

*After 3 to 4 months' storage in the sludge chambers.

At Essen-Nord, where the sludge is extraordinarily voluminous, it was desirable to determine the moisture content of the fresh sludge, and therefore it was necessary to use special apparatus for sampling. This apparatus proved to be very well adapted to the sampling of fresh sludge for any purpose. A description of it is here given:

A glass funnel, A, containing about 15 liters, is provided with a faucet at about one-third of its height from the bottom. The lower, pointed end of the funnel is provided with an outlet made of a perforated cork, glass and rubber tubing, and a clamp. The funnel rests upon a wooden stand over a receptacle for the sludge. Every hour the funnel was filled with sewage and the sludge allowed to settle to the bottom. At the expiration of the hour the faucet, H, was opened and the supernatant liquid allowed to drain off, when the funnel was again filled with fresh sewage. Finally, when the sludge had filled the funnel to above the faucet, H, this was again opened and all sludge and sewage above it allowed to escape, when the outlet, K, was opened and the remaining sludge received into the container, S, and sent to the laboratory for examination.

Moisture in the Sludge

The percentage of moisture in the fresh sludge was not determined at Recklinghausen, Schwerin or Holzwickede. This was done, however, at Essen-Nord, where the moisture content was found to be 94.6 per cent. average. The moisture content of the decomposed sludge has been tested at all plants of the Emschergenossenschaft continually since they have been in operation. For Essen-Nord the average figure is 84.4 per cent. The weight of the decomposed sludge is, therefore, but 34.5 per cent. of the weight of the corresponding fresh sludge. Here it is well to make note of a popular fallacy. Many engineers in thinking of a reduction in moisture percentage from, say, 90 per cent. to 60 per cent. consider that the volume of the 60 per cent. sludge is 25 per cent. of the volume of the 90 per cent. sludge. This is, however, incorrect. The weight of the 60 per cent. sludge is 25 per cent. of the weight of the 90 per cent. sludge. The volume has been reduced to less than 25 per cent. of its former value, because the specific gravity of the dry substance is greater than the specific gravity of water, and the greater the ratio of dry substance to water the greater is the part it plays in determining the volume. Suppose, for example, the specific gravity of the dry substance is 2. Then the volume of the 60 per cent. moisture sludge is 21 per cent. instead of 25 per cent. of the 90 per cent. moisture sludge. Of course the relative difference between volume and weight is much less significant when the moisture contents are high, say above 80 per cent.; but as shown in the foregoing, the difference is considerable when comparing 90 per cent. and 60 per cent. sludge. However, despite this large decrease, the moisture content of the decomposed sludge at Essen-Nord must be regarded as abnormally high, as must also that at Recklinghausen, namely, 83.4 per cent. The reason for this high moisture content is the same in both cases. Both sewages receive trade wastes (Essen-Nord especially iron salts) which cause voluminous hydrates to settle with the sludge, and thus keep the water content high.

At plants, however, where pure domestic sewage is received the moisture content is much lower. The following table contains the percentage moisture in the decomposed liquid sludge and in the drained spadable sludge fom the sludge-drying beds of seven different disposal plants:

Table 6—Percentage Moisture Content of Sludge

Plant	Decomposed liquid sludge	Dried spadable sludge
Aplerbeck	77.2	65.4
Essen-Nord	84.4	66.5
Holzwickede	72.2	56.0
Recklinghausen	83.4	65.2
Schwerin	75.3	50.0
Sodingen	77.1	47.6
Westhausen	70.3	47.6

It will be noticed in plants like Essen-Nord and Recklinghausen, where the moisture content is abnormally high in the decomposed liquid sludge, that the drying on the sludge beds removes a correspondingly high percentage of this moisture content, so that the end products of the various plants do not differ in moisture content nearly so much as might be expected from the differences in weight of the liquid decomposed sludge. To make this clear, the accompanying sludge diagram has been prepared from the data of the above table. This diagram also shows clearly the relative decrease in weight of fresh sludge, liquid decomposed sludge, and dried spadable sludge on the basis of one ton of dry substance in each case.

It is evident from the data of the foregoing experiments that there were differences in the degree and nature of decomposition at the various plants. This is no doubt due not only to the differences in the fresh sludge, but also to the fact that the period of decomposition in one plant was different from that in another.

Remarks Upon Utilization.

The following methods of sludge utilization seem worth considering:

1. Use as fertilizer.
2. Recovery of fats.
3. Production of illuminating gas.
4. Use as fuel.

In considering the value of sludge as a fertilizer the quality or character of the sludge is as important as the quantity of nitrogen. The potassium in the sludge cannot be changed by the process of decomposition, as it is mineral in character. The phosphorus content may be subject to some change, but as it is small in quantity the value it adds to sludge as a fertilizer is unimportant. The previous tables show that there is less nitrogen in decomposed sludge than in fresh sludge, but probably as much or more nitrogen is actually available in the decomposed sludge than in the fresh sludge. This is because the fresh sludge contains a greater amount of fatty and fibrous material which renders the earth more or less impervious to air and moisture, and probably does more harm than it does good as fertilizer. It is as a result of this fact that sewage farms in the past have experienced "sick fields." The decomposed sludge, on the other hand, contains much less fat and is not fibrous. In addition, the fat which does remain is very finely divided and uniformly distributed through the mass of the sludge; and since the sludge itself is porous on account of its gas content, this finely divided, well-distributed fat is not an appreciable hindrance to percolation. The sludge drains quickly, remaining porous when dry. Hence, when it is used as a fertilizer the porosity of the earth is not affected, so that air and moisture can penetrate everywhere, and come into contact intimately with the nitrogenous material, and subsequently with the roots of the vegetation. It is probable, therefore, that despite its lower nitrogenous content it is a better fertilizer than fresh sludge. Because of its spongy, porous character it may also be used profitably in cases where the ground is "heavy."

The economic recovery of fats from sewage is at the present time possible only where particularly large quantities of fatty materials are discharged into the sewers. This is, therefore, but seldom. However, where the recovery of fats does come into question it is probable that here too decomposed sludge would be cheaper to handle than fresh sludge.

The tables show much less fatty material in the decomposed sludge than in the fresh sludge; but it is more than likely that the increased cost in handling the latter material, because of its sticky impenetrable consistency, more than offsets the increase in fats recovered. It should be remembered, too, that the foregoing experiments were made upon the dried solid matter of the sludge, and that for the same amount of dried material the volume of fresh sludge is from two to three times, or more, the volume of decomposed sludge. This means, of course, that for all practical purposes the cost of handling the fresh sludge, with reference to the amount of dried material treated, is two or three times as great as the cost of handling decomposed sludge. Therefore, even if the fats in the decomposed sludge were only one-half to one-third as much in amount as the fats in the fresh sludge, the cost of getting the same amount of fats into the recovery apparatus would be the same in either case. In the four sets of experiments above described we have the following percentages of fresh sludge fat remaining in the decomposed sludge: Seventy-three per cent, 44.3 per cent., 62.8 per cent., 45.3 per cent. The lowest figure shows but a little less than half, while the higher figures show almost two-thirds and three-fourths. Hence, in view of the above remarks as to relative volumes we may confidently say that since the decomposed sludge can be much more easily treated than the fresh sludge, one volume of fat will be recovered more cheaply from the former than from the latter.

Fats in Sludge

It is to be emphasized here that the fats still contained in the decomposed Imhoff-tank sludge are not referred to as an advantage, but, on the contrary, as merely a lessened disadvantage. The practical-minded engineer would much prefer a total removal of the fats in the sludge, because in most cases it is hardly to be doubted that the value of the recovered fats is less than the decrease in disposal costs produced by their absence. In addition, if the sludge is considered from the standpoint of use as a fertilizer the total absence of fats would be a decided advantage.

In considering sludge as a material for illuminating gas production it is necessary to consider the relative amounts of volatile matters. From the tables it will be noted that the same amount of dried matter of decomposed sludge contains one-half to three-fourths as much volatile matter as the same amount of dried fresh sludge. Here again, however, it is necessary to keep in mind the relative volumes of the two kinds of sludge for the same quantity of dried material. Not only does so much more material have to be handled in the case of fresh sludge, but practically two or three times as much water has to be evaporated or otherwise removed.

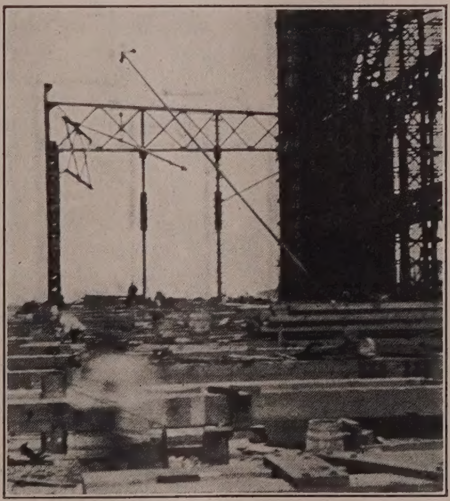
The fixed carbon in sludge undergoes only little change in amount through the process of decomposition, and therefore in considering the relative merits of fresh and decomposed sludge as fuel, the relative volumes to be treated and the relative amounts of water to be removed per unit of fixed carbon are the important factors. It is obvious that the decomposed sludge is the cheaper to handle.

In addition to all of the above considerations a very important one for all kinds of utilization is that of transportation wherever the sludge-utilization plant cannot be constructed upon the site of the sewage-disposal plant. Even if the process of decomposition reduces the moisture content only from 90 per cent. to 80 per cent. the amount to be transported will be reduced by half.

It is not claimed that all of the possible departments of sludge utilization have been considered, but it is believed that the examples quoted will illustrate the general truth that within certain limits the question of sludge utilization does not depend so much upon the quantities of valuable elements contained as upon the amount of moisture and the physical character of the sludge. It may confidently be stated that in most cases the cost of recovery for fresh sludge will be so great as to place the whole question of utilization beyond economic limits, whereas it may be possible in some of these cases with decomposed sludge to derive at least some profit. Therefore, in comparing two different sewage-disposal designs, one of which deals with fresh sludge and the other with decomposed sludge, it should not be taken for granted that the latter method will yield a lessened profit. The foregoing article intends to show that for any decomposed sludge corresponding with Imhoff-tank sludge the profit will probably be greater.

A Building-Erection Kink

In the erection of Machinery Hall, the large timber structure for the Panama-Pacific Exposition at San Francisco, described in the Engineering Record of Aug. 30, one of the booms proved to be too short for placing a truss at a corner point. The difficulty was overcome by the method shown in the accompanying photograph. A hanging lever was suspended from the end of the boom, the truss was placed on one end of the lever and a chain was attached to the other end. By swinging the boom and manipulating the lever from the ground by means of the chain the truss was easily swung into place.



Lever for Lengthening Boom

Destruction of Gamboa Dike

The Gamboa Dike protecting the Culebra Cut of the Panama Canal will be dynamited Oct. 10, according to the "Canal Record." All available steam shovels are now working on the rock sections in the bottom of the canal between the Cucaracha slide and the Empire suspension bridge. The greater part of the remaining slide material is to be removed by dredges. Rock berms will be drilled and charged after the water is admitted, so that the material may be blasted with the water as a cushion and later removed by dredges.

The City Tunnel and Conduits of the Catskill Aqueduct

Abstract of Paper Read before New England Waterworks Association by Walter E. Spear, Department Engineer, Board of Water Supply, City of New York

The Catskill water supply is to be delivered within the city limits of New York by a large pressure tunnel nearly 18 miles in length, which is being constructed in bed rock through the heart of the city. Such a system of distribution is unique in waterworks construction. The task of finding sufficient room in the streets for the number of 48-in. and 66-in. mains necessary to carry 500,000,000 gal. of water per day was quite impossible. The streets in which many of these trunk mains would necessarily have to be laid were already occupied by sewers, pipes, and ducts including the large trunk mains of the present water-supply system. The subways also interposed an effective barrier to the laying of large water mains for the transportation of the Catskill supply. Aside from these physical difficulties and the great inconvenience and loss which the public would suffer from constructing these mains in the streets, the expense of delivering the Catskill supply in steel or cast-iron mains was found to be excessive, and the deep pressure tunnel plan was therefore adopted. It was readily demonstrated that such a tunnel, with frequent outlets, having a capacity equivalent to the full yield of the Catskill sources, could be carried deep in the bed rock under the city at half the expense of laying an equivalent number of cast-iron or steel mains, and that this plan was in every way superior to a system of pipes in the streets.

Description of Tunnel

The tunnel is entirely under public parks and streets except for 1027 ft., or 1.1 per cent of the total length. All but four of the shafts are likewise located in parks or streets, so that but little real estate had to be acquired by the city for the project. The depth of the tunnel for fully half its length is about 200 ft. below the surface. The city tunnel is to perform the functions of a trunk distribution main. It is not to be simply an aqueduct delivering its entire flow to some objective point, since this tunnel is to have waterway shafts at intervals of about three-fourths of a mile, through which water in large volumes may be discharged directly into the distribution mains. The tunnel has a finished diameter varying from 15 ft. at the upper end to 11 ft. at the lower for the first 7.7 miles.

Of the twenty-four shafts through which the tunnel is being constructed, twenty-two of them are to be finished as waterways. No. 1, the first within the city limits, is to be filled upon the completion of the tunnel; and No. 11 is to be a drainage shaft through which the northerly half of the tunnel may be unwatered. Provisions for draining the southerly half of the tunnel are being made at Shaft 21, but this shaft is also to have a waterway.

One of the most interesting features of the city tunnel is the design of these waterway shafts. The diameter inside of the outer lining of concrete masonry is 14 ft. From a point 100 ft. below the rock surface, the section of the shaft is to be constructed to a diameter of only 4 ft. within a concrete-lined steel riser pipe, solidly concreted into the shaft. This riser is to be capped at the top, within the large masonry valve chamber just below the surface of the ground, by a bronze tee having two 30-in. outlets, to each of which are to be attached in tandem two gate valves, one a bronze-mounted service valve and another of solid bronze placed next to the tee. The latter valve, which may be closed in the valve chamber or from the surface above the chamber, is intended to be used when the service valve is out of order. At most of the shafts, on by-passes about the service valves, regulating valves are to be installed by which the pressure on the distribution side may be controlled to meet the service requirements.

In addition to these valves at the top of the riser pipe, an emergency valve of the needle valve pattern is to be provided in the shaft at the foot of this channel, which is designed to close automatically when the discharge through the riser exceeds a certain velocity; the valve can also be closed by hand either from the chamber or from a control on the surface at some distance from the shaft. All but three shafts are of the circular type. Of these exceptions, Shaft 21, the drainage shaft, is approximately egg-shaped, to include both the 14-ft. drainage well and the standard 48-in. waterway; and Shafts 13 and 18, which are section valve shafts, are roughly rectangular to provide space for a well over the section valve in the tunnel and for two 48-in. waterways, taking out of the tunnel on either side of the section valve. The section valve, according to the design, is to have a clear waterway of 62 in. and the tunnel is to be contracted on either side to this diameter in long, tapered transition sections to minimize the loss of head. The purpose of these section valves is primarily to permit of cutting off and draining the southerly portion of the tunnel south of Shaft 13 or 18 while still operating the northerly portion.

The amount of water delivered to the city is to be measured by a Venturi meter placed in the main tunnel just north of Shaft 2. The throat of the meter is designed to have a diameter of 8.67 ft. with flaring sections on either side, through which the transition is made from the 15-ft. tunnel section.

No project was ever more thoroughly investigated than the city tunnel, and the rapid progress which has been made in driving the tunnel and the freedom from serious difficulties which has been experienced have more than justified the comparatively small expense of the preliminary work; altogether 201 borings, aggregating 46,343 ft., were made and over two years spent on surveys and studies before contracts were finally prepared.

CONSTRUCTION

Contracts for the city tunnel were let in four sections and work was begun in June, 1911; it is expected that the entire work will be completed on schedule time ready for the delivery of water into the city's mains by the latter part of 1915. At the present time, after twenty-six months' work, the city tunnel is 70 per cent completed; 95 per cent has been excavated and 20 per cent lined. One of the four sections of the tunnel is now so far advanced that it will be substantially completed inside of six months.

No great delay was experienced in starting the excavation of the shafts in rock except on the southerly section, where the rock surface is too far below the ground surface to permit of excavating through the drift by other than pneumatic methods. Reinforced concrete caissons 15.3 to 18 ft. in interior diameter, with walls 2 to 3 ft. in thickness were sunk to rock and tightly sealed at these locations.

From two to three months were consumed in sinking the shaft caissons and in sealing them, from the time the steel shoes were set up in the bottom of the chamber excavation, and maximum air pressures from 17 to 46 lb. per square inch were used. So carefully was the work done that no appreciable settlement that could be attributed to the caisson work was observed in the vicinity of the shafts.

The depths of the rock shafts of the city tunnel range from 156 to 714 ft., and, with the exception of the timbered section valve shafts, the outer concrete lining was generally placed in stretches of 100 ft. as the shafts were excavated. From 2.5 to 7.7 months were taken in sinking and lining the shallower shafts in the neighborhood of 200 ft. in depth and from 6.5 to 12.5 months for the deeper ones, from 250 to 450 ft. in depth. Shaft 21, which has a larger cross-section than the other concrete-lined shafts, and a total depth in rock of 714 ft. to the bottom of the drainage sump, took 14.5 months. The best month's record on all contracts in sinking and lining was 112 ft. at shaft 20, the average about 50 ft. ranging at the several shafts from 25 to 82 ft.

The first tunnel headings were turned at shaft 14 in Central Park in December, 1911, and the last at shaft 21 in February, 1913. At present, 95 per cent of the entire tunnel has been excavated, leaving less than 5000 ft. to be driven. The top heading and bench method common in this country has been generally used in excavating the tunnel. Pneumatic drills have been generally employed both in shafts and tunnels, although some extensive trials of electric drills were made by one of the contractors. Except for the northerly section of the tunnel, which is largely in undeveloped sections of the city where a central steam-driven compressor plant could be erected, each shaft, or, in one case in Central Park, a group of three shafts, was equipped with a compressor plant of one to three electrically-driven units. All shaft hoists, ventilating fans, and most of the pumps in the tunnel, are also electrically driven as required by one of the clauses of the tunnel contracts, to minimize the noise and dust incidental to the operation of steam plants. Among other measures adopted for the safety of the public as well as the workmen in the tunnel was the storage of the explosives at each shaft in a large underground magazine excavated in the rock at tunnel grade and connected with the tunnel by a tortuous drift, driven laterally from a point 100 ft. from the shaft. A heavy steel and timber door, hung in a concrete bulkhead at the entrance to this drift, was designed to protect the workmen from the gases of combustion.

No records in tunnel driving or in shaft sinking have been broken on the city tunnel. The routine and methods usually employed in carrying on mining operations in the country have had, of course, to be materially modified in doing such work in the heart of a large city. The best progress thus far made in excavating the tunnel has been that in the Inwood limestone in the south heading of shaft 20, where 354 ft. of completely driven tunnel of 11-ft. finished diameter were excavated in one month. The average progress on the several sections has been from 150 to 250 ft. per month, according to the difficulties encountered and the speed which the contractors considered it necessary to push this part of their work.

The lining of the tunnel is well under way on only one section. The methods of concreting in the city tunnel differ little from those which have been developed on other tunnels of the Catskill aqueduct, though a somewhat richer concrete, mixed in the proportions of 1 of cement to 1½ of sand or screenings and 3 of stone or gravel, is being used in the city. First the concrete of the invert for a width of 5 to 6 ft. is laid against radial form boards. This part of the concreting goes rapidly, since from 150 to 200 ft. may be readily placed in an eight-hour shift. It is customary to lay all the invert in any section before starting the next operation. After the invert is finished the sidewalls are concreted to the springing line, and finally the arch is placed. The last two operations have come to be done simultaneously, the sidewall being concreted with an adjacent section of arch. This plan permits the mixing plant to be run at full capacity during the slowest and most difficult part of the concreting, the keying of the arch, the surplus concrete going to the sidewall form. Most of the concrete lining in the city tunnel, thus far put in, has been placed in sidewall and arch forms 60 ft. in length, all in tunnels of 13 and 14 ft. in finished diameter, but forms 80 ft. long are being successfully used in the 12-ft. tunnel. Greater lengths of form are being attempted in the 14- and 15-ft. tunnels, but it has not yet been demonstrated that these forms can be successfully filled at any considerable distance from the mixing plant. The 60-ft. forms are usually concreted every third day, or twice each week, and on the average 1300 ft. of completed arch and sidewall in the 14-ft. tunnel have been concreted each month from one plant with three sets of 60-ft. forms. Metal forms of the Blaw type are being used throughout.

COST OF THE TUNNEL

The city tunnel when completed will have cost in the neighborhood of $20,000,000, including the contracts for equipment, and all real estate, engineering, and other expenses. This amount corresponds to a unit price of $216 per foot of tunnel, and in addition to the tunnel itself includes the cost of over 8000 ft. of waterway shafts, together with the masonry chambers at the surface and the equipment.

Beyond the terminal shafts of the city tunnel, pipe conduits are being laid for the supply of the outlying sections of the city. The connection across The Narrows to Staten Island or the Borough of Richmond is to be a 36-in. flexible joint pipe laid in a dredged trench with a cover of 8 ft. and placed at no point within the pier head lines less than 45 ft. below mean low water. The total cost of these trunk conduits in the city, including engineering expenses, is estimated at $2,000,000.

Preventing Forest Fires

No damage from forest fires has been reported this season in the coast district of British Columbia. Only three fires occurred in the entire district patrolled by the fleet of wardens launches, and these were put out before they had syspread beyond the "slashes" in which they started. The good record this year is attributed largely to the efficient organization of fire wardens by whom trails have been cut and lookout stations and telephone systems maintained. In previous years the province has been obliged to pay heavy bills for the extra labor required in emergencies. The "fighting force" fund, as it is called, exceeded $20,000 for the Selmo fire in 1910, it is reported, while the damage done by this fire amounted to $750,000. This year the fighting force fund has been drawn on for less than $200.

Illinois State Bridge Specifications

Comprehensive specifications for concrete and steel highway bridges, prepared under the direction of Mr. A. N. Johnson, state engineer; Mr. T. R. Agg, road engineer, and Mr. C. Older, bridge engineer, have been issued by the Illinois Highway Commission. They contain important changes from previous editions of the specifications, particularly in regard to impact allowances, reversing stresses in steel bridges and proportioning gravel concrete. Specifications for creosoted material for bridge floors have been added. The specifications will cover all bridge work contracts under the direction of the commission. Besides the contract forms and the usual clauses, general definitions, instructions and requirements, unit stresses for different members are specified in considerable detail for tension, compression, shearing, bending, bearing, reversing and for field connections under twenty-six different values, eight of which are for compression, most of them specifying a fixed amount of 16,000 or 18,000 lb. per square inch diminished by 60, 70 or 80 l/r, where l is the length and r is the least radius of gyration, both in inches.

The paint requirements are very specific and require that samples shall be weighed and when possible the clear vehicle drawn off and tested. The remaining vehicle is then to be extracted four times with 88 deg. gasoline, washed once with benzole and once with ether. When the vehicle cannot be drawn off, one-half of the sample will be extracted directly with gasoline, benzole and ether. One hundred minus per cent of pigment found will be considered as the per cent of vehicle. The determination of volatile oils shall be made by passing a current of live steam through a flask containing 50 drams of the vehicle or of the sample maintained at 100 deg. C. until 100 cu. cm. of water have been distilled over. The volatile oils which have passed over, together with the water, will then be separated by a separatory funnel.

Letters to the Editor

Federal Aid for Highways

Dear Sir: By a confusion in regard to the meaning of the terms "Federal aid" and "national highways" the editorial article in the Engineering Record of Sept. 6 inadvertently misrepresented the aims of the National Highways Association.

The advocates of national highways believe in practically the same program as that of the Charlotte (N. C.) "Observer," quoted in your editorial. They believe that the national government should build a system of trunk lines connecting the principal cities of the country, entirely at the government's expense and under government control; that the states should build certain secondary roads entirely at the states' expense and under state control; and that the counties should build a third class of roads entirely at counties' expense and under the county control.

The advocates of Federal aid for road building believe that the government should go into partnership with the states and pay either a per cent or a fixed amount per mile for various types of hard roads; that the government should turn its money over to the states and let the states spend it.

The National Highways Association realizes with you that some form of national participation in roadbuilding is inevitable and that the thing to do now is to guide the movement into sane channels. The writer believes also, as you do, that "highway officials and engineers who refuse to be stampeded into the rabid Federal-aid camp would do well to work together." To facilitate their working together instead of pulling in different directions is one of the aims of the National Highways Association. The writer believes that under Federal aid the waste, extravagance, and graft would be appalling—a thousand times greater than under the systems of "county aid" and "state aid," which in most states have been discarded as unsuccessful experiments and have been superseded by out-and-out county road building and state road building.

Since the terms "Federal aid" and "national highways" involve the basic principles of highway construction there can be no intelligent discussion of the road question without a clear understanding of the meaning of the two terms. In view of the fact that in your editorial the National Highways Association was made to appear an advocate of Federal aid, I shall appreciate your correcting this error and giving due prominence to the real position of the association.

Charles H. Davis,
President, National Highways Association.
South Yarmouth, Mass.

[The Charlotte "Observer" said, as quoted in this journal Sept. 6, that it "would welcome and advocate Federal aid as auxiliary to these state efforts." Its position, therefore, is diametrically opposed to the policy of the National Highways Association and not practically the same, as Mr. Davis states.

While the Engineering Record is glad to have Mr. Davis make clear the position of his association the statement does not in the least alter this journal's position as to the association's aims. Rather it intensifies the opposition. The great "pork barrels"—the river and harbor bills and the public buildings bill—are not of the "Federal aid" order but are out-and-out "national" projects, paid for entirely by the Federal government and executed under its sole control. Yet they have been a stench to the noses of those with any notions of economy in government. "National highways" may not be "Federal aid highways," as Mr. Davis construes the terms, but to the Engineering Record's thinking they are "Federal aid highways" of the most objectionable sort.—Editor.]

The Sherman Law and Agreements between Engineers Not to Enter Competitions

Dear Sir: On page 112 of your issue of July 26 Mr. Walter H. Drane's letter contains a suggestion that would be a violation of the Sherman anti-trust law, if put into effect to apply beyond the boundaries of any particular State. The Sherman law reads as follows:

"Section 1. Every contract, combination in the form of a trust or otherwise, or conspiracy, in restraint of trade or commerce among the several States or with foreign nations, is hereby declared illegal. Every person who shall make any such contract or engage in any such combination or conspiracy, shall be deemed guilty of a misdemeanor, and on conviction thereof, shall be punished by fine not exceeding five thousand dollars or by imprisonment not exceeding one year, or by both said punishments in the discretion of the court." (26 Stat. L. 209.)

Mr. Drane's letter was in part as follows:

"The writer would like to suggest an addition to the code of ethics, and this in the interest of the dignity of the profession. It is that the members of the society agree not to enter into any competitive contest in supplying plans. The writer has taken part in several farces of this kind and he believes that this foolish practice is responsible for the failure of many engineering structures. At the present time he has on file a set of plans for a small bridge. The bridge commissioners met and under the persuasion of contractors agreed to have competitive plans. The writer argued in vain that this really was no competition, as not a member of the commission knows one thing about bridge design. The engineers entered into it eagerly. The commissioners failed to see that as the writer was upon salary and got no fee for his plans he had no inducement to cut the bridge to dangerous limits, while the scheme adopted offers every temptation to others to do that very thing."

Any such agreement as proposed by Mr. Drane would be useless unless extended to interstate competition. The fact that Mr. Drane's letter was published outside the State is evidence of that intent. Such an agreement, therefore, extended across a State boundary would be a direct violation of the law. Moreover, many of the States have even more drastic anti-trust laws to govern competition inside the State.

Mr. Drane seems to view ethics in too restricted a sense. A code of ethics for any profession should define the duties of its members for the greatest good of the entire profession.

In the bridge competition referred to Mr. Drane should remember that there were numerous civil engineers in the employ of contractors who would be benefited by the opportunity to submit competing plans. These engineers employed by contractors are usually trained specialists and as much entitled to consideration as Mr. Drane himself.

In fact, is it not Mr. Drane's failure to recognize his own proper function that is the cause of his complaint? Mr. Drane was apparently employed by the commission as their consulting engineer; that is, as their technical adviser to help them secure the most efficient bridge. Instead of accepting that capacity he insists upon acting as their designing engineer, a position properly filled by a specialist in bridge design. A commission of laymen properly advised by a competent consulting engineer needs have no hesitancy in securing real competition by including competition in design along with labor and materials.

It is a striking commentary on the inefficiency of the civil-engineering profession that it has not yet solved that simple problem of selecting the most efficient design for any proposed structure. It is very easy to secure a comparison of designs, but almost impossible for the average civil engineer to compare designs in conjunction with the actual cost of each. This is the one most to be desired thing that clients ask of civil engineers, yet for how long will the profession refuse to solve the problem?

By "failure of engineering structures" Mr. Drane doubtless meant failures by collapse. An engineering structure may also be a failure by reason of excessive strength and waste materials. The failures by collapse are always detected and invariably punished, but the other kind are rarely found out. For this reason their numbers are infinitely greater, their losses more serious, and additionally encouraged by the lack of competition that Mr. Drane commends.

If a code of ethics is to be devised let it be broad enough to cover the entire profession and not for the benefit of the favored few. And let it not be forgotten that the interests of the profession will be served only when the best interests of the clients whom that profession serves are kept in mind.

Indianapolis. Daniel B. Luten.

Pond-Week Fallacy

Dear Sir: The resentment which is caused in this locality by your editorials on the subject of the Missouri good-roads days and Kansas pond-week is obviously not intentional, but due to the fact that you write them at a distance, unaware of the true conditions of affairs here.

The main idea of your editorials is that good-roads days and pond weeks are to be deplored on account of the great waste of energy which such sudden concentration of enthusiasm is apt to produce. Permit me to tell you a little more about what a Missouri good-roads day is really like.

As an engineer assisting in the direction of this work I spent two days in covering as much of one county in Missouri as was possible by automobile. Although in the course of the two days I saw one gang of fifteen men who were idle for about an hour on the morning of the first day for lack of direction, and one piece of road machinery which had been misused, until we arrived to correct this misuse, and one culvert out of several hundred which was about to be built in the wrong manner and upon which possibly two hours' work had been wasted, the great confusion and lack of energy and direction which your editorials imply were nowhere evident.

The reason for this is that in every county in Missouri there is in existence a good-road building organization, which awaits only the necessary funds and proper enthusiasm to stimulate it to action. In every county the road work is under the direction of a county court all the time, usually through its county engineers, or in a few instances through township officials, under the advice of these engineers. Every county is divided into small districts in charge of a road overseer, who lives in that district and who knows the nature of the roads, and is selected on account of his knowledge of them. The fact that his position is a political one does not prevent him from being a good roadbuilder.

Now when Governor Major issued his proclamation the county court simply called a meeting of its thirty-six road overseers and organized them especially for the work to be done on the two days designated. This organization was materially assisted by the business men, who served with enthusiasm on various committees to stir up and direct such interest.

From the business men they solicited funds; from the machinery men they solicited motor cars, road machinery, tools, dynamite, and the like, and succeeded so well that all these were on the job at 8 o'clock on the morning of the first day, or before.

The material men, with these motors at their disposal, furnished the necessary vitrified pipe, and so on, for culverts and other structures. Under this broad co-operation the work throughout the county went forward better than many a contract job I have noted; and what was true of the one county which I observed was undoubtedly true of all the other counties in Missouri, for they all had the same organization.

One of the most valuable of all aids was the co-operation of the newspapers, which not only advertised the undertaking widely beforehand but acted as clearing houses and headquarters for all parties concerned, instead of indulging in mere criticism.

Aside from all theories as to what could have been accomplished by other means, or what could not have been accomplished by this means, the result of the two days' good-roads agitation was undoubtedly a great improvement to a great many hundred miles of road, to say nothing of the augmented public interest in road work.

Now, when it comes to building ponds in Kansas they ought to have equally good results. First of all, the extraordinarily dry spell of the summer just past has made every farmer that has not got a pond want one. In the rich, rolling counties of Kansas there are thousands of places where a little energy, well directed and supplemented with a few slip teams, would produce ponds all the way from 1 to 10 acres in extent, each one of inestimable value to the owner.

There is nothing new in the idea of ponds on the farm, except that nowadays, following the example of the old-time Missourian, J. H. Christopher by name, we are no longer content simply to throw up a mud dike for the water to overflow in some freshet and wash away, but we provide each substantial dam with a diversion or underflow pipe made out of ordinary vitrified clay sewer pipe, so that, having risen to the maximum level of safety, the water is drawn off through this underflow pipe just as it is drawn from a bathtub before it fills and floods the premises.

In conducting this work Governor Hodges will have the assistance of the engineers at Manhattan, where there is located one of the best engineering schools in the country. Prof. L. L. Dyche, the State Fish and Game Warden, will also assist, and why, under these conditions, a pond week in Kansas should turn out to be a "fallacy" is not easy to foretell by one who has watched the real success of the Missouri good-roads days.

Kansas City, Mo. Benjamin Brooks.

Fire Disasters and Automatic Sprinklers

Dear Sir: The writer has noted your editorial in the issue of Aug. 2 referring to the Binghamton fire disaster. It strikes me that some of the strongest points you could have made have been missed in that you did not call attention to the use of the automatic sprinkler as the means of preventing both fire and panic, and, therefore, safeguarding lives in a more effective way than could possibly be done by any mere means of getting people out of the building. The fact that the sprinklers would, at the same time, completely safeguard both the building and its contents, as well as the lives, cannot be lightly regarded.

In the August number of the "Journal" of the American Society of Mechanical Engineers the case is very well put. Particularly is the point made clear that every time a sprinkler operates from fire in a building full of people it is a potential life saver.

The writer has been studying this subject of automatic sprinklers and their use in saving both property and life for some little time. So far as he has been able to learn there have been just eight deaths by fire in factories protected by automatic sprinklers. In not a single one of the eight cases could the death be charged to the failure of the sprinklers to operate properly, as you will see by considering the details.

In January, 1907, under peculiar circumstances, a fire gained great headway in the Cocheco Mills, Dover, N. H., at a time when the automatic-sprinkler system had been temporarily shut off for adjustment. This was a cotton mill, and the spread of the fire was so rapid in the cotton goods on the machines and up through beltways, etc., that five of the employees, who attempted to save their belongings in their lockers, lost their lives. The sprinklers were not operative at the time the fire broke out. Before they could be adjusted and put in service the damage had been done. This is, of course, a strong argument in favor of keeping all such equipment at all times ready for immediate service.

About a year ago a celluloid-comb factory near Boston was the scene of a fire which flashed all around the room in the scraps of extremely inflammable celluloid on the work benches, making a complete circle of flame, which shut off access to the stairs. About a dozen employees dashed through the flame and down the stairs, all being more or less severely burned, two of them losing their lives. Before the fire department arrived on the scene the automatic sprinklers had the fire completely out. It would, of course, be too much to have expected the employees to remain within the circle of flame for the sixty or ninety seconds required by the sprinklers to put out the fire. Probably no human being under such circumstances, and with the panic which is always connected with a fire, would have remained at this time. But it is perfectly clear that if all the employees had remained in the room the chances are that none of them would have been severely injured.

The other case where life was lost under automatic sprinklers was in a piano factory in New Jersey, also about a year ago. The watchman's lantern exploded while he was making his rounds and set fire to the inflammable varnishes in the room through which he was passing. He was so severely burned as a result of the explosion that he died the next day. The sprinklers held the fire from spreading, giving the firemen an easy task in putting the fire out.

Practically every expert who reported upon the results of the Asch Building fire in New York two years ago said that automatic sprinklers would probably have prevented the loss of life in that building. As you probably noted, the automatic sprinklers have since been installed throughout that building.

Cases are on record where a fire in a wastebasket full of paper has been put out by a sprinkler before the wastebasket was consumed. The sprinklers operate without human assistance, are not subject to panic conditions or excitement, are not hampered by smoke or a sudden flash of flame, and are not hindered in their work by a rush of frightened human beings running away from the seat of disturbance.

To my mind they are the only great saving factor in our fight against fire, and should be required, legally, in all manufacturing plants subject to destruction. They can be put into existing buildings at nominal expense and render what is now a firetrap more safe against both panic and loss of life than any "fireproof" building now existing which does not contain sprinklers. If you consult the real-estate advertising pages of the daily press you will find building after building designed for loft and manufacturing purposes making capital of the fact that they are "protected by automatic sprinklers."

Boston. Sidney G. Koon, M. M. E.